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A Ferry-Boat Propelled by the Current

By C. E. Chappell*

Civil engineering has been defined as "the art of directing the great sources of power in Nature to the use and convenience of man." With this definition, the ferry described below should be an example of good engineering, since it derives its motive power directly from the stream over which it operates.

The Cumberland River at this point flows north in a broad curve to the left, throwing the thread of the current near the right or east bank on which is situated the City of Clarksville, Tenn. The ferry-boat is fastened by a wire cable 1500 ft. long to the west abutment of the Louisville & Nashville R.R. bridge, and the cable is supported by five buoys. Some idea of the arrangement can be gained from Fig. 1.

The ferry itself is a flat-bottom barge 18 ft. wide by 54 ft. long, decreasing in depth at the ends, and will carry five teams at a trip. On the upstream side is a movable vane or leeboard about 3 ft. in depth, which may be lowered to give the current a greater bearing against the side of the boat. The vane is controlled by the left-hand wheel of the two shown near the center of the barge in Fig. 2.

About the axle of the right-hand wheel, the middle of a 1½-in. rope is wound four or five turns, and the two ends of the rope are carried over sheaves near opposite ends of the boat to connect with the steel cable at a point 30 or 40 ft. upstream. By turning this wheel the ferryman can throw one end of the barge upstream, causing the current to strike against the side of the boat and propel it across the stream in the arc of a circle to which it is held by the steel cable.

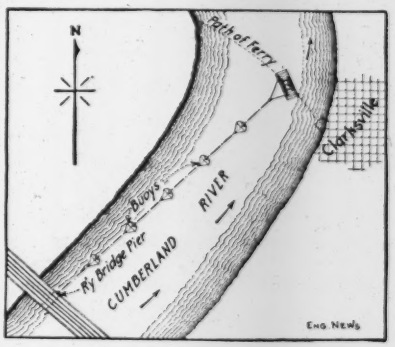

Fig. 1. Diagram to Show Operation of Current Ferry

Fig. 2. Ferry-Boat Moving Toward Farther Shore
(The current is from right to left.)

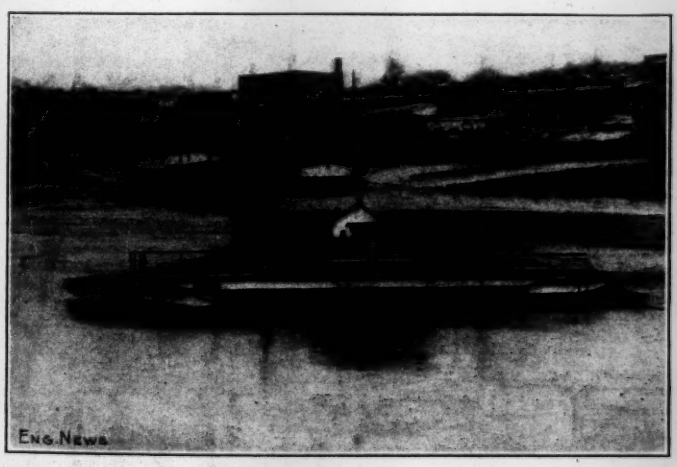

Fig. 3. One of the Buoys, Showing Manner of Attaching Cable

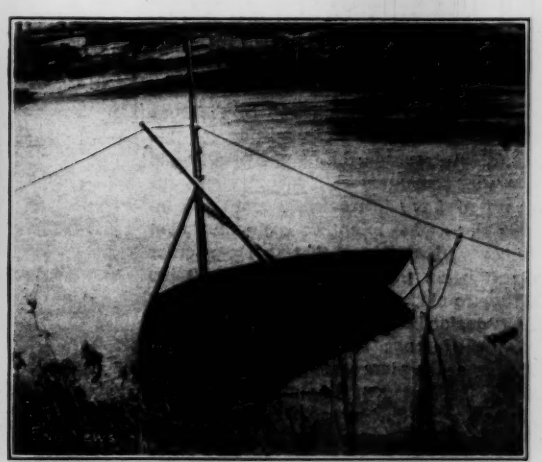

Fig. 3 is a nearer view of one of the buoys and shows the manner of supporting and attaching the cable. Besides the mast supporting it, there is a diagonal arm which causes the buoy to respond to any movement of the downstream portion of the cable. On the upstream side, the buoy is attached to the cable by bridle lines slack enough to allow some turning. Soon after the barge leaves the shore the long straight side of each buoy is turned at a slight angle from perpendicular to the current and is consequently propelled across the stream in the same manner as the barge itself.

The ferry is operated from 6 a.m. until sunset at all stages of the river up to 45 ft., the approaches being at quite steep grades down the river banks. As the water rises, the steel cable is taken up enough to land the ferry where the approaches reach the water's edge. The river is from 600 to 750 ft. wide at this point, depending on the stage.

A new boat is required every six years and costs about $600. Buoys last longer and cost $50 each. Several years ago the ferry was purchased from the toll company by the county for $6000, and is now operated as a free ferry. The total cost of operation, including interest on the investment, salary of ferryman, depreciation and repairs on boats and approaches is about $1500 per year. A bridge over the river at this point would cost not less than $150,000.

The fixed, operating and maintenance charges on a bridge then would easily rise to ten times the present annual cost of the ferry, and it is very improbable that the traffic could rise correspondingly to warrant the expenditure.

*Clarksville, Tenn.

Remarks on the Quebec Bridge, and a Proposed Cantilever Design

By C. A. P. Turner*

The writer was much interested in the recent articles† by Gustav Lindenthal on the Quebec Bridge, and his exceptionally interesting design of an inverted braced arch for the structure. He is inclined, however, to believe that, if a fair comparison is made of the relative merits of the cantilever and the inverted arch, the cantilever presents for the consideration of the engineer the more desirable structure.

The writer is inclined to agree with the general conclusions of the Board of Engineers for the Canadian Government in the following particulars: (1) that the cantilever type of structure is to be preferred; and (2) that a single-intersection design is to be preferred and should be more economical than a double-intersection. He disagrees with them as to the details of carrying out such a design, and agrees with Mr. Lindenthal that the design finally adopted does not compare favorably with the famous Forth Bridge.

The writer further agrees with Lindenthal in his condemnation of the conduct of the so called competition for this work. As regards the height permissible for the structure, the specification appears to have been drawn not with an idea as to what might be economical, but rather with the idea that this particular specification would of necessity limit the structure closely or practically to the outline decided upon by the board. The height of truss adopted for the Forth Bridge at the piers was 336 ft. on centers. while the Board of Engineers arbitrarily limited their design to 290 ft., or 50 ft. shallower, for a structure carrying heavier loads. This entailed a large increase in the weight of metal without materially reducing the overturning moment.

The shore arm of the original design of the board is much longer than economy would dictate, and this lack of proper proportion gives the official design a peculiarly clumsy and ungainly appearance. But the view of one of the engineers of the board was that, the structure being so large, it could not be otherwise than ugly. The writer believes that the majority of the engineering profession would take issue with this view.

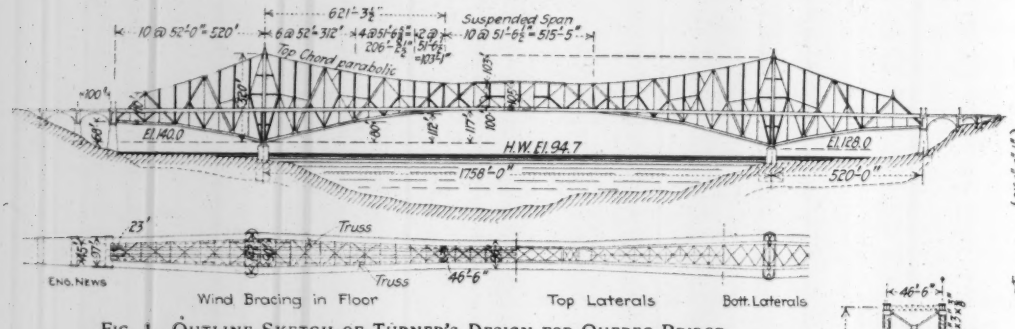

FIG. 1. OUTLINE SKETCH OF TURNER'S DESIGN FOR QUEBEC BRIDGE

It cannot be expected that any two engineers would arrive at once at the same conclusion or opinion regarding a structure of such unusual proportions, and this is one of the reasons why the writer has taken the trouble to present for the consideration of the readers of ENGINEERING NEWS the outcome of some little study made by himself on a design for this structure. The design which resulted from this study is presented below, and some drawings are given. Before taking it up, however, some general remarks are offered.

THE FUNDAMENTAL QUESTION OF T... AND COMPRESSION MEMBERS

The failure of the original ... Bridge demonstrated two important ... sitions in bridge design: '

First, that for a heavy tension ... no form can be devised which wi... sent so little uncertainty as to it... gree of security and strength as a chain of well made eyebars.

Second, that in the make-up of compression members stiffness and rigidity in all directions is an essential requirement, if we expect to develop the value of the material of the section.

No one is inclined for a moment

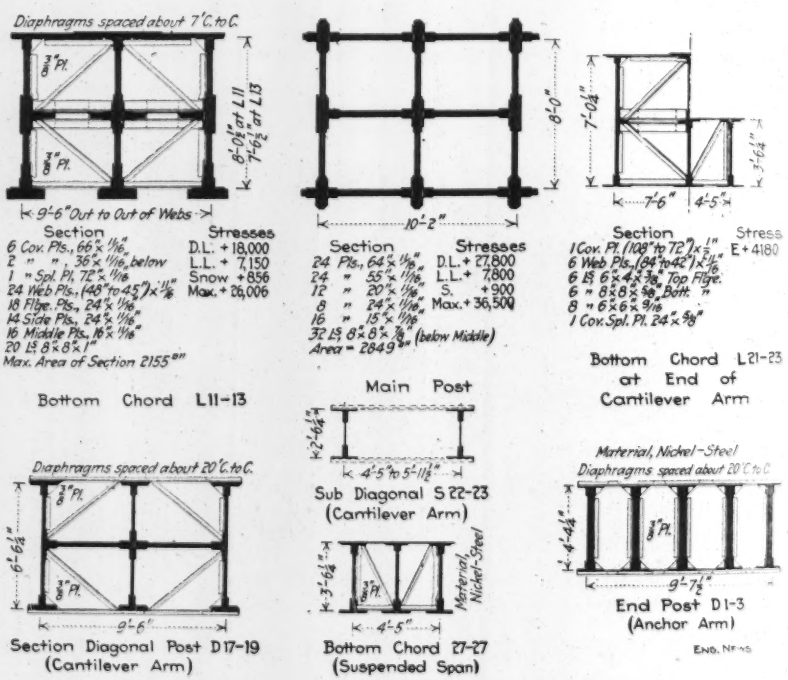

FIG. 2. SECTIONS OF COMPRESSION MEMBERS, TURNER'S DESIGN FOR QUEBEC BRIDGE

*Consulting Engineer, Phoenix Bldg., Minneapolis, Minn.

†Eng. News, Nov. 16 and 23, 1911.

to question the absolute security and strength of the compression members of the famous Forth Bridge. Many engineers, however, are inclined to question the degree of strength presented by the built-up tension members in this great structure: To what extent would the riveted joints develop under test the strength credited to them by the designers of this structure?

In the official design by the Board of Engineers for the new Quebec Bridge, the demonstrated superiority of the eyebar seems to have been appreciated and recognized. The board's eyebar chords were double, however, and involved the assumption of a high order of workmanship in boring for their respective connections before it would be permissible to assume that the two sections making up the chord were actually working together in perfect unison.

This departure from preferable form was explained by the huge section needed in the chords of a structure of the capacity required by the specification. But by the adoption of an economical depth over the main pier, and suitable details, there seems to be no trouble whatever in packing the entire chord on a single pin.

The second proposition demonstrated by the failure of the original Quebec design seems to have been recognized only in a limited way by the members of the Board of Engineers in working up their design for the proposed structure. In the original Quebec design, the metal was concentrated in the webs with little or no flange. In the official design of the board, while a slight improvement in this respect was made, yet the proportion of metal which might be considered flange was insignificant compared to the amount of metal incorporated in the webs, instead of being in the neighborhood of 50% as should be the case in a well designed section.

As a result of this make-up, small test models of expensive nickel-steel developed no better values than we should expect to secure with the same grade of metal used in the Forth structure provided the metal was distributed on more rational lines.

General Description of Turner's Cantilever Design

The design is for a central span of 1758 ft. and employs a shore-arm length of 520 ft. The suspended span is made 515½ ft. long. For neatness of appearance and for economy in securing anchorage, concrete approach arches are used. The height of truss over the main piers is fixed at 320 ft., with regard to proper economy of design.

The design contemplates building the structure by cantilevering out to the middle of the suspended span, rather than floating the suspended span in (which is too costly with a span as long as economy dictates). The closing of the upper chord

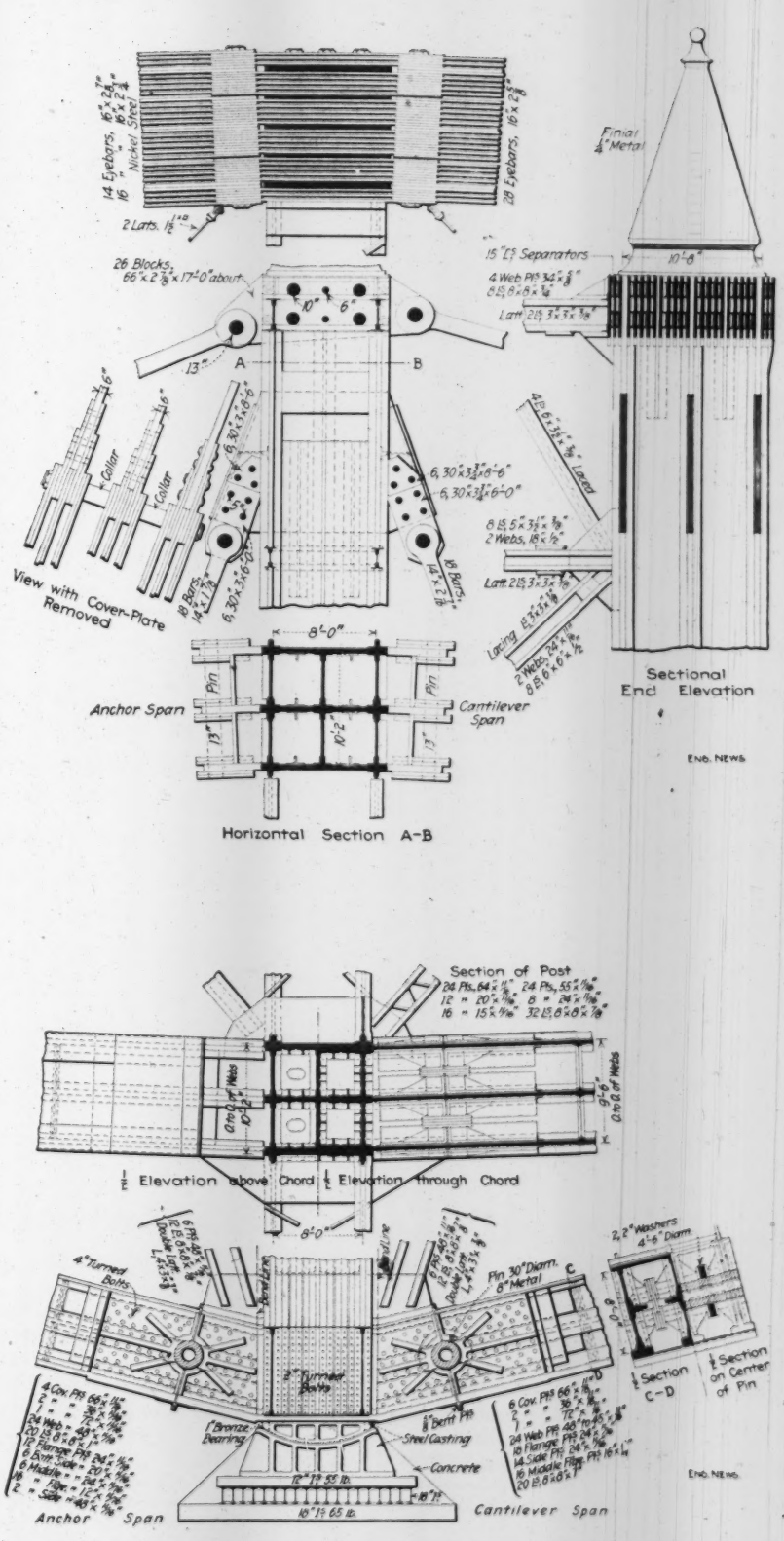
Finial 4" Metal
15 "L" Separators
4 Web Pls 34" x 1/8"
Lats 2 Ls 3" x 3" x 7/16"
26 Blocks, 66" x 2 1/8" x 17'-0" about
28 Eyebars, 16" x 2 3/8"
14 Eyebars, 16" x 2 3/4" x 16" Nickel Steel
2 Lats. 1/2" x
Collar
Collar
View with Cover-Plate Removed
18 Bars, 14" x 2 1/8"
18 Bars, 14" x 2 1/8"
6, 30" x 3 1/4" x 8'-6"
6, 30" x 3 1/4" x 6'-0"
Laced
8 Pls 5" x 3 1/2" x 7/8"
2 Webs, 18" x 1/2"
Lats 2 Ls 3" x 3" x 7/16"
Lacing 2 Ls 3" x 3" x 7/16"
2 Webs, 24" x 1/2"
8 Pls 6" x 6" x 1/2"
Sectional End Elevation
Anchor Span
Cantilever Span
Pin
Pin
8'-0"
Horizontal Section A-B
Section of Post
24 Pls. 64" x 1/2" 24 Pls. 55" x 7/16"
12 " 20" x 7/16" 8 " 24" x 1/16"
16 " 15" x 1/16" 32 Pls. 8" x 8" x 7/8"
Webs
Webs
Elevation above Chord
Elevation through Chord
8'-0"
4" Turned Bolts
Pin 30" Diam. 8" Metal
2" Washers 4'-6" Diam.
2" Turned Bolts
1" Bronze Bearing
Steel Casting
Concrete
Section C-D
Section on Center of Pin
Anchor Span
Cantilever Span

FIG. 3. BASE AND CAP DETAILS OF MAIN POST, TURNER'S DESIGN FOR QUEBEC BRIDGE

is to be done on permanent wedges. The upper toggles and bottom wedges at the end of the cantilevers, for making the central closure, are to be operated by electric motors.

The width of the lateral trusses is made about 140 ft. at the main piers; this is even greater than in the Forth Bridge (120 ft.). The Forth Bridge has its trusses battered toward the top. In the present design the trusses remain vertical, but converge toward each other from main pier to anchorage and from main pier to suspended span. They are spaced 90 ft. apart over the main pier, 45 ft. at anchorage, and 46½ ft. at the suspended span. The greater width of the wind truss is obtained by placing the roadway outside the trusses and using the roadway chord as wind chord.

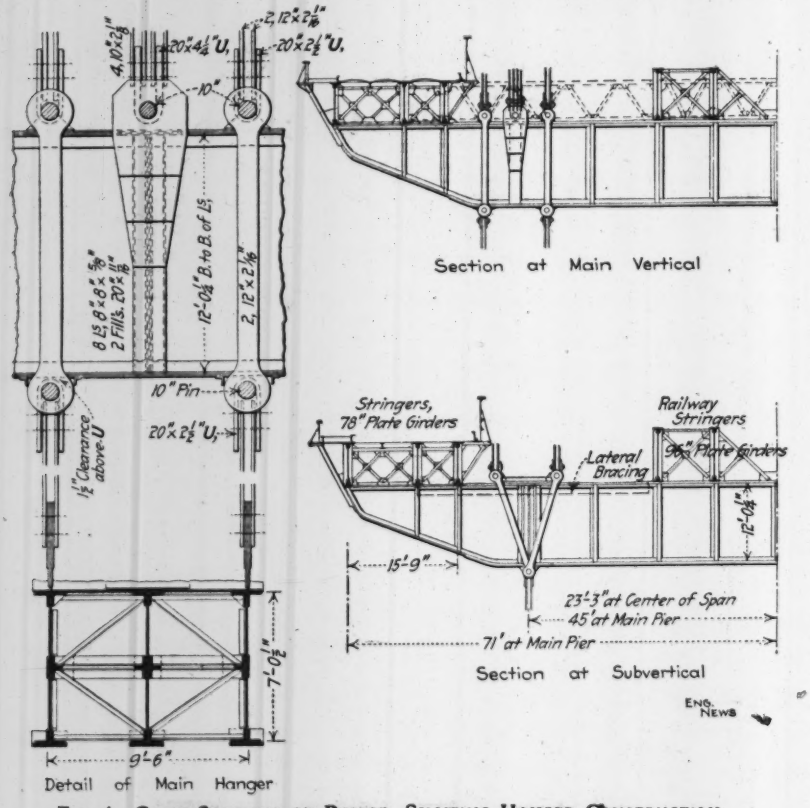

FIG. 4. CROSS-SECTIONS OF BRIDGE, SHOWING HANGER CONSTRUCTION. TURNER'S DESIGN FOR QUEBEC BRIDGE

WEB SYSTEM—Mr. Lindenthal has called attention to the fact that both the official design of the board and the accepted design of the St. Lawrence Bridge Co. are lacking in economy as compared with the Forth Bridge. The reason for the failure of the official board and the accepted design to secure this economy is of interest. As the writer looks at this question, the lack of economy lies in the multiplicity of main connections to be found in the accepted design of the St. Lawrence Bridge Co., and in the failure of the Board of Engineers to select an economical diagram for the official design.

Comparing the cantilever design accepted with the one suggested by the writer, we find that the top chord, between the main vertical post over the pier and the anchorage, has seven main joints in the accepted design where diagonals and top chord intersect, against two main joints in the writer's proposed design. Between the main post and the suspended span the accepted design has eight such joints against three in the writer's design. Each joint of this kind involves a large amount of metal and runs rapidly into weight, besides adding to the complications of erection.

Although the main diagram is laid out with wide meshes, sub-chords are used for convenience in erection and to assist in distributing the panel-load concentrations into the web.

The diagram outline of this proposed design was worked out primarily from the standpoint of ease of erection and simplicity of details. Rather than allow the desire to obviate secondary stress dictate and determine the character of the diagram, the question of provision for secondary stress was left to be provided for by mechanical ingenuity.

The displacement diagrams of the design (taking into consideration the dead-load after the structure is completed) would indicate large secondary stresses in the diamond frame over the main piers, unless mechanical provision were made to reduce them. Such reduction is accomplished by wedge-pin adjustment of length in the sub-posts forming the lower part of the diamond; as the main post shortens or the ties lengthen, the sub-posts can be adjusted in length so that the sub-chord or horizontal tie crossing the central portion of the diamond is not overstrained. The main shoe details, comprising a hinge, complete this provision from the mechanical standpoint.

It will be noted in the diagram selected that a large part of the dead weight is concentrated directly over the main pier. The huge vertical posts and bracing, weighing approximately 4000 tons, produce no bending moment whatever in the truss proper. In contrast, the official design by the Board of Engineers has its heavy posts arranged on diagonal lines over the pier, involving much difficulty of erection and necessitating heavy temporary members in order to place the finished structure in position.

ERECTION APPLIANCES—In erecting a structure of this type, provision must be made for handling and delivering the material as the work is built out. In the Phoenix design of the Quebec Bridge, heavy girders were attached to the bottom chord, projecting outside of the trusses, and a traveler spanning or straddling the trusses was used. With the proposed design this method of procedure would not be practicable, and the writer's idea was to adopt a mushroom type of traveler supported directly from the steam-railway tracks, and deliver the truss material as the bridge is built outward on the roadway or electric-railway tracks outside of the trusses. In this manner the material may be delivered at practically the point from which it is desired to raise it, and, with the overhanging traveler stayed to the ends of the floor-beams, we have thus a simple and comparatively light frame from which to handle the work. The weight of the 1000-ton traveler used by the Phoenix Bridge Co. in erection was made up in large part of the heavy members needed to support the traveler. Eliminating these, as we can readily do through the method of support outlined, the weight of the traveler would be much less than half that which it was necessary to provide for in the original Phoenix design.

ARRANGEMENT OF FLOOR—In the proposed design the roadway and the street-railway tracks are kept outside of the trusses, and the railway tracks are placed between trusses. By this arrangement the dead weight of the roadway counterbalances in a large measure the dead weight of the railway, which reduces the bending moment in the floor-beams very materially indeed.

WIND CHORD AND BRACING—By arranging the buckleplate flooring of the roadway in such a manner that it is riveted to the longitudinal stringers, we have a wind chord of unusual section and

rigidity, which with a fair amount of diagonal bracing furnishes a wind truss independent of the chord lateral bracing and lying directly in the plane of the floor.

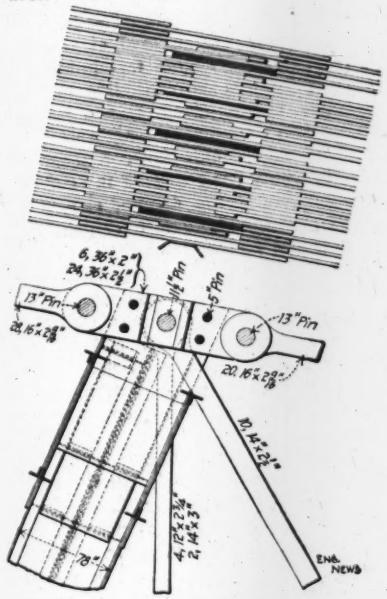

FIG. 5. TOP CHORD JOINT, TURNER'S DESIGN FOR QUEBEC BRIDGE

Heavy lateral bracing is provided between the compression chords, while the wind stress in the upper system is provided for largely by transverse or portal bracing.

IMPORTANT DETAILS

COMPRESSION MEMBERS—The make-up of the compression members is simple in the extreme. Three webs are used throughout the main chords, and there is ample room to work inside. Typical sections of chords and other compression members are shown in Fig. 2.

The characteristics of the chord make-up are: Provision for approximately 40% of the section in the flange; cover-plates covering the top of the heavy section, well riveted to heavy flange angles connecting the webs; no cover-plate whatever for the bottom of the box, but wide flange-plates, giving a heavy flange (24 in. in width) at the bottom of each web; concentration of metal in the center of each web and the stiffening of the whole box by heavy diaphragms spaced about 7 ft. apart. The construction gives in effect a cellular member of such ample dimensions that the work of riveting up splices, etc., can conveniently be carried on therein.

The make-up of the main posts, like the make-up of the main compression chords, presents approximately the same stiffness in both directions. The post is made up of three webs in the plane of the truss, and a transverse web connecting the centers of these. The webs (parallel to the truss) are each provided with heavy flanges well latticed, a combination which gives a high degree of stiffness under bending from wind pressure, in view of the unusual width of the member, and renders the connection to the upper and lower chords a very simple matter indeed.

To secure an economical as well as esthetical design of trusses, the upper chord eyebar chain is hung on a parabolic curve and the anchor and cantilever-arm lower chords have each two breaks in direction. The lower-chord members are tapered both in width and in depth from the main pier toward anchorage and suspended span.

The section of the lower half of the main posts is 2849 sq.in. The lower chord of the anchor arm in the panel adjacent to the main post has a section of 2155 sq.in., and the maximum section of the cantilever-arm chord is 1951 sq.in. All these members are of carbon steel. The upper chord in the sections extending from the main post contains 1348 sq.in. on the anchor-arm side and 1176 sq.in. on the cantilever-arm side. This chord is made up of nickel-steel eyebars.

FLOOR HANGERS—The suspension of the floor from the panel-points of the truss is worthy of notice. In the hangers dropping from the main panel-points of the top chord the suspenders are made up in two sections independent of each other, one section taking care of the floor-beam loading directly beneath the panel-points, and the other section continuing to the lower chord and taking care of the truss dead load plus such portion of the live load as is transferred through the sub-system.

These suspender connections, an unusual feature, are illustrated in Fig. 4. The great width of the chord, varying from 8 to 12 ft., necessitates a wide separation of the strands of the main hangers. These are arranged by a spreader to be brought to a central pin, so that there is no eccentricity in the application of the floorbeam load nor will any flexure of the floorbeam under live-load affect this distribution in the least.

CAST-STEEL CONNECTIONS—In an ENGINEERING NEWS editorial regarding the Phoenix design of the Quebec Bridge, attention was called to the difficulty of making a satisfactory pin joint without excessive metal in details unless by far the major portion of the section was distributed through the webs. The details presented in the writer's proposed design

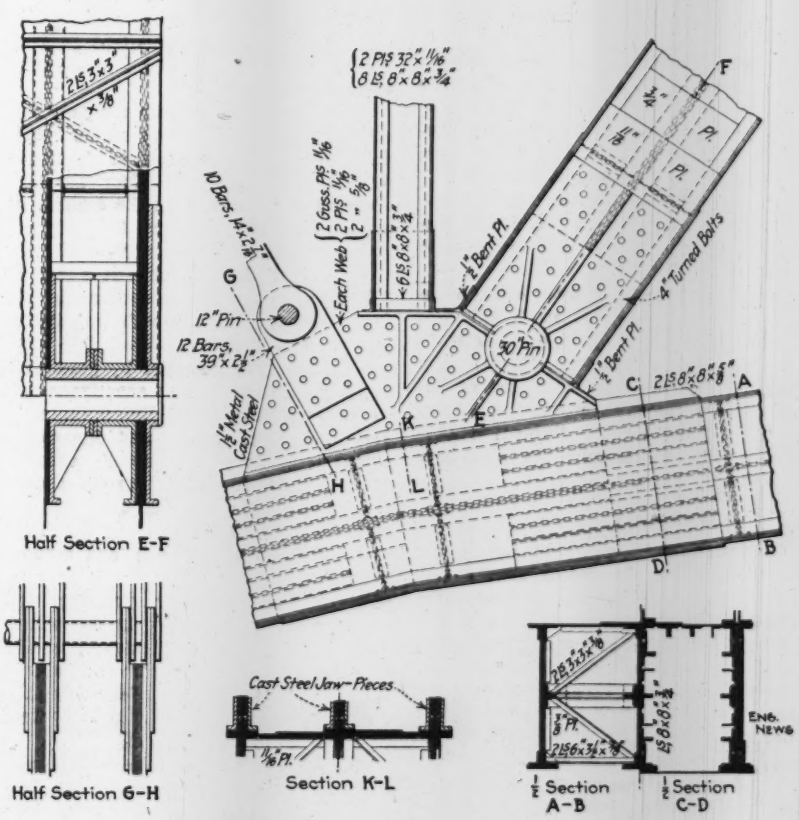

FIG. 6. BOTTOM-CHORD JOINT WITH CAST-STEEL PIN-JAWS. TURNER'S DESIGN FOR QUEBEC BRIDGE

place these joints entirely outside of the section of the bottom chord, and the forked compression member is practically eliminated. This is accomplished by reinforcing the gusset-plates, which extend above the chord to receive the web members, by cast-steel pin-jaw blocks, as exemplified in Fig. 6. However novel this may be, the detail is not only safe, but also—by avoiding the difficulties above mentioned—distinctly economical.

The same method of reinforcement was employed at the bottom-chord joint over the main shoe. The six cast-steel jaw pieces on each side of the splice between post-gussets and chord provide ample bearing for the pins (Fig. 3).

TURNED BOLTS IN PLACE OF RIVETS—These heavy connections were made practicable by the rather radical innovation of substituting large turned bolts for the rivets. At the splices over the main pier and in most other cases, 4-in. bolts are used, but in some joints 2-in. bolts. They are to be fitted tight to their holes, and forced in by hydraulic jacks; their nuts are then to be screwed up tight so as to pinch the plates firmly together. The writer believes this to be a decidedly novel scheme for making connections.

DOUBLE-PIN LINK CONNECTIONS—The top-chord connections of the web necessarily involve forked ends on the compression members. These forks, however, are never longer than three or four times the thickness of the metal. The design of these top-chord joints was simplified effectively by the link construction illustrated in Fig. 5.

MULTI-PIN CONNECTIONS—For certain connections, a group of small pins was laid out, in place of a single pin or riveting. These pins are usually 5 in. in diameter, but in some cases pins of different sizes are used together. The details of the post-cap and diagonal connection, in Fig. 3, illustrate multi-pin connections.

RIGIDITY AND WEIGHT

DEFLECTIONS—The computed maximum displacement at the center of the suspended span for E-75 loading was 17½ in. For maximum one-sided loading it was estimated that the middle top-chord section of the suspended span would be swung 14½ in. out of line, assuming the lower chord to be held true to line by the floor system and the heavy lateral system in the plane of the chord. An interesting feature of the one-sided loading is the live-load stresses set up in the sway bracing connecting the main posts, due to the convergence of the trusses.

WEIGHT—In Mr. Lindenthal's article he discusses the possibilities of a cantilever for Quebec, and I regard his discussion as quite conservative, taking exception only to his view that the double-intersection or multiple-system web is economical. Mr. Lindenthal figures that with the double-web system a design should be gotten out with 56,000 tons of steel, using increased working stresses. Using the working stresses provided for in the specification of the board the writer's estimate of weight was approximately 10,000 tons less than the estimate of the board, and this with a saving of nearly $1,000,000 worth of nickel, as nickel-steeel was figured on only for the eyebars and some special details.

What Is the Future of the Oil Engine in the United States?

BY G. H. KIMBALL*

The development of the Diesel oil engine has been pronounced by one of our leading inventors to be one of the seven greatest achievements in the world. From the European countries there are coming to us every day reports of the activity of their engine builders in the development of engines of this type. In this country, on the other hand, the Diesel engine has been exploited by one company with rather uncertain success, although there have been a large number of engines sold.

Lately, however, another company has entered the market with an engine that is an improvement on the type furnished by the older company, since the new design tends to cut down the number of parts. The older company for their largest engine produced a machine, with six 16x24-in. cylinders, capable of a load of 450 b.hp. The new competitor in the field presents a machine with four 21x26-in. cylinders that will deliver 600 b.hp.

It is generally admitted by prominent engineers and verified by experience that the fuel economy of the Diesel engine is very high at all loads; and one would expect it to be more generally used in power plants, for oil is far more convenient to handle than coal and there are no ashes to be disposed of, and the boiler room, with its added labor and repairs, can be done away with. The fact is, however, that in every manufacturing plant steam is necessary in the winter months and in many plants all of the time, so if the exhaust steam can be used for this purpose, the cost of operating the steam engine is much less than to operate those driven by oil fuel.

But in electric central stations and pumping stations, there is a wide field open for this form of prime mover, offering as it does such signal advantages that nothing but water power can compete with it in cheapness of operation.

In any installation up to 1500 kw. capacity, the oil plant would require fewer men for operation than a steam plant of the same capacity. Repairs, however, would amount to about [illegible] times as much in the oil plant as in the steam, and one unit would need to be idle at least 10 hours out of the 24 in order to keep them all sufficiently overhauled to carry their rated load. For it must be remembered in the case of the internal-combustion engine that it has no reserve of power corresponding to that obtainable in the steam engine by increasing the pressure or lengthening the period of admission. Since a plant of this size would need four of the 600-hp. units or five of the 450-hp. size, it is well to note the many parts contained in one unit and how systematic the care of them must be. The auxiliary equipment would need to be amply adequate, for the operation of the engine depends upon its supply of compressed air and no less important is the cooling water supply, so that no part of the installation can be neglected if even ordinary good service is desired.

As this type of engine is made very heavy and is vertical in design, there must be plenty of room overhead with a good hoisting apparatus of not less than three-tons capacity and more would be better. Every facility should be afforded for quick and efficient repairing and the work must be done with exacting regularity. No other system will suffice and it has been well borne out by experience that those Diesel engine plants which have been a success have followed a strict régime in the matter of the upkeep of the machinery and have been willing to hire good and conscientious engineers and pay them good wages. It might be said that this same program should be followed in any steam plant; to which statement I would agree, but whereas the well designed steam engine will operate for a year and possibly more with very slight adjustments, the Diesel engine must be overhauled several times during that period.

As the oil engines are made today, there are not only a large number of units necessary for a comparatively small plant, but in each unit there is a great number of parts to be kept watch of, many of which will wear and cause trouble. For this reason there has been great difficulty in getting competent operators who will take the necessary interest and exert themselves to get good results.

We have had the figures regarding costs of steam installations placed before us many times, designers can figure how long will be the life of the machinery, and the science of steam-engine operation has attained a very high standard. With the oil-power plant, on the other hand, we are venturing on a field that has plenty of warnings of prospective troubles ahead but few good reports of success, for while there are some plants which have been running nearly ten years, there are no reliable data regard-

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ing the cost of repairs and the condition of the engines today.

The operators who have made a success of running the Diesel oil engines are very few, while there are many who have injured not only the engines themselves but the reputation of this type as a reliable prime mover. I have been asked by steam engineers if these engines would run on, day after day, so that any dependence could be placed upon them; and they have been surprised at my answer that for a year's continuous running, in my experience, there never has been a failure to start and run all day. It must be remembered, however, that the owners of Diesel power plants cannot afford to have their engines always under the eyes of expert testers of machinery. Therefore an engine must be produced which will be rugged in construction.

The large gas engine is now fairly successful and it is made in sizes nearly as large as the steam units in all except the very largest central stations. For the oil engine to invade this field it must also be made in larger units than any that have been thought of as possible by American builders. This brings up the question of how much the size of the cylinders can be increased so that not more than three cylinders will be required for a 2000-hp. engine. This seems like a very great increase, but marine engines approaching that horsepower are building in European countries and why should they not be built here?

It is worth noting that while the first engines built in this country with six 16x24-in. cylinders and a speed of 164 r.p.m. developed 450 hp., as far back as 1908 a German builder had produced one with three 21½x26-in. cylinders that developed 500 hp. at 155 r.p.m.

After nearly three years' experience in charge of oil-engine plants, it became evident to me that for the small station the many parts requiring attention and adjustment and the frequency with which they must be overhauled was a serious handicap upon their use, just as it is more obviously a barrier to their use in the large station. The largest Diesel engine plant in the world is located in the southern part of the United States and has eight six-cylinder units with a combined capacity of 3600 kw. The amount of repairs and supervision necessary to keep such a plant in proper working condition is appalling and no other plant as large has been attempted since.

The German engine referred to above had some features that I call very good and worthy of being imitated by the American builders. First, its speed was not excessive, being lower in fact than the American engine. It also had the cranks open to observation, in contrast to a system of splash lubrication. The principal objection to inclosed crank cases is that there is a temptation to leave them too much to themselves. The oil soon becomes mixed with carbon particles that work out around the pistons and cover everything with a layer a quarter of an inch thick. After this has occurred, the parts are in poor condition for a thorough inspection; and in fact it is a very unpleasant task to do any work in such close quarters even if there were no oil to be cleaned off. Again, if a bearing has been adjusted too tight there is no way of getting any warning until it is too late.

With the open crank case, the cylinder can be oiled with a force-feed lubricator, the cranks and bearings can be supplied by a pipe system with good engine oil which can be filtered and always kept clean. This feature should do much toward decreasing the friction losses of the inclosed crank-case type.

Another good feature in the German engine was the crosshead and guides, which must do much to keep the piston from thrusting so hard against the cylinder walls and put the wear where there is a chance for adjustment.

There has been much trouble with the governors on the American engines on account of their unsteady motion, which seemed to be in time with the strokes of the fuel-pump pistons and may have been due to the pull of the valve rods on the camshaft. I have for some time advocated driving the governing mechanism directly from the crankshaft and the latest American builder has adopted this arrangement.

It is now customary to put every valve in the cylinder head, so that all of them must be disturbed if the head is removed. I think it would be very advantageous if the piston could be gotten at by the removal of the head alone, as is done on steam engines, the valves being placed in ports cored out on the sides of the cylinders. This would make the inspection of the pistons a matter of much less time than is now required.

Since the principle of the engine is so simple, their ability to carry the rated load continuously depends upon the tightness of the valves and the pistons. It is a small matter to keep the former in good shape, but it is much more difficult in the case of the piston rings, as the wear is excessive and the carbon collects in a short time, binding the rings firmly in their grooves. For this reason it is very necessary that they be examined frequently. As the cylinders are apt to wear as well as the rings, they often need reboring. The bushed cylinder is now being manufactured which should make the matter of replacing of a cylinder less expensive.

In the above remarks I have tried to bring to the attention of men interested in these engines the things which in my opinion, after close observation of the engines as produced at present, seem to affect them adversely. I have done this not with a desire to criticize but to stimulate the interest that is slowly growing in a lower priced fuel.

Up to the present, many features have been improved. The engine has been made more reliable in starting and reasonably steady running, the speed regulation is now very good and a well made machine has been produced; but to successfully compete against the large steam plants those things which have been mentioned above will have to be attained. Many experiments on engines for marine work are being conducted and there is now being constructed in England an oil engine that is identical with the Diesel in principle, with the exception of the high compression of 500 lb. per sq.in., this being reduced to 150 lb., and the ignition made by means of a hot tube. In consequence of this, the air pressure for injecting the fuel is not over 500 lb., whereas 1100 lb. is necessary with the higher pressure. This is an engine working on the two-stroke cycle. If this design should prove feasible, much larger cylinders could be constructed.

Another very important feature is that the parts of the future engines must be easily accessible, so that the vital parts may be readily examined. With engines combining the above features there is no reason why they should not be used extensively in the United States.

A Plan for the Electrification of Berlin city and suburban railways has been laid before the Prussian Diet, according to an item in a recent issue of "Engineering" (London). The electrification of the city and suburban railways of Berlin has been contemplated for some years. The number of passengers carried on Berlin city railways rose during the period from 1885 to 1909 from 75,000,000 to 157,000,000, while that on the suburban railways increased from 41,000,000 to 127,000,000 during the same period. Assuming the same rate of increase during the next seven years, it is estimated that the number of passengers on city and suburban lines in 1916, when it is expected that the electrification work will be completed, will exceed that of 1909 by not less than 103,000,000. The present steam traction system is altogether unable to handle this traffic.

The new electric trains are to consist of 12 to 13 large cars with an electric locomotive at each end of the train during the periods of heaviest traffic. The energy is to be distributed from an overhead line carrying alternating current at 15,000 volts. A private company has undertaken to supply the necessary current from two power stations—one at Bitterfeld and another in the immediate vicinity of Berlin. The stations will be constructed for an output of 100,000 kw. each in the first place, with provision for subsequent enlargement. The transmission cables will be placed underground.

It is claimed that the state loses money every year on the Berlin local traffic on account of the extreme cheapness of the fares. The loss in 1911 is given as some $500,000. In connection with the introduction of electrical traction, an increase of the fares is contemplated.

Viaduct Approach of Cellular Type

By S. W. Bowen*

A cellular type of construction for a reinforced-concrete viaduct is somewhat of a novelty in concrete work, and is described in this article. Additional interest is given to the article by the details of cost and by the comparison of cost of this and other types of construction.

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During the summer of 1908, the Manufacturers' Ry. Co., of St. Louis, built a single-track viaduct extending from the storage yards on the levee near Dorcas St. to the tracks lying west of Second St. The viaduct crosses two streets and a large number of railway tracks, and was built in order to give a railway connection between the upper and lower yards which would be free from the inconvenience and danger of grade crossings. The total length of the structure, including approaches, but exclusive of the earth embankments, is about 2156 ft. A portion is on a 15° curve, the central angle being about 100°. The maximum grade on tangent is 2%, while that on the curve is 1%.

The viaduct proper consists of steel-girder spans, with steel columns and bracing, supported on concrete pedestal piers with reinforced-concrete piles. At each end of this steel structure there is an approach consisting of two parallel reinforced-concrete retaining walls, connected at intervals by thin concrete walls in which tie-rods are embedded. The space between the walls is filled with cinders. The north approach consists of 1100 ft. and the west approach of 200 ft. of this type of construction, while the length of the steel structure is about 856 ft.

Owing to the large number of crossings, the steel portion of the viaduct consists largely of through spans. There were but two short stretches where deck spans could be used. Economical span lengths could not be used because of the arrangement of the tracks to be crossed. The steel structure was designed under Cooper's specifications of 1906, and for the E-50 loading.

Foundation Conditions

Borings were made at intervals along the line of the steel structure in order to determine the elevation of bed rock. This was found to lie not over 21 ft. below the ground surface, west of the tracks of the St. Louis, Iron Mountain & Southern Ry. on First St., but it dipped rapidly to the east of these tracks. The west abutment of the steel structure over the tracks of this railway is supported on concrete shafts extending to rock, and all column footings west of First St. also extend to rock. The concrete piles under a few of the column foundations east of First St. extend to rock, but the majority of the column foundations and the north abutment of the steel structure over Dorcas St. are supported by concrete friction piles, and by spread bearings on the soil. The foundation loads were assumed to be equally divided between the piles and the soil.

In order to get some idea of the safe bearing value of the piles and soil, tests were made by driving a pile, loading it with rails and taking levels on it from day to day. The pile was allowed to project about a foot above the ground surface and was capped with 12x12-in. timbers which supported the 85-lb. rails used for loading. At two points along the line, pits were dug 3 or 4 ft. deep, and a platform having a bearing surface of 4 sq.ft. on the ground was constructed in each pit. These platforms consisted of four 12x12-in. timbers, about 5 ft. long, bolted together to form a block 2 ft. square. This block was set on end in the pit, and was covered with a platform of plank, on which 85-lb. rails were laid. The pile sustained a total load of about 50 tons before appreciable settlement occurred, while the first observed settlement of the platforms took place at about 2600 lb. per sq.ft. pressure on the soil. As a result of these tests, it was decided to use 1000 lb. per sq.ft. bearing on the soil, and 25 tons per pile. The soil in this vicinity is silt deposited by the river, overlaid by a cinder fill of varying thickness, and is not capable of sustaining heavy loads, especially when saturated. The column footings were spread considerably as a result of the low bearing pressure used, and had to be reinforced to properly distribute the loads.

The concrete piles are of the molded and driven type. They are reinforced with longitudinal rods and spiral winding wire, the reinforcement amounting to about 1⅛% of the volume of the pile. The piles are 25 ft. long, of hexagonal cross-section, and taper from 14 in. to 8 in. short diameter. They were driven with a steam hammer, using a patented air cushion.

Cellular Approach Construction

The reinforced-concrete approaches are of somewhat unique design. Each consists of two parallel walls, connected at intervals by cross walls. The side walls were built in 25-ft. sections, with expansion joints between them to do away with the temperature stresses on the longitudinal reinforcement. On tangent, there are two cross walls for each 25 ft. section of structure, spaced 20 ft. apart, so that the distance between the cross walls of adjoining sections is 5 ft. On the curved portion of the structure an additional cross wall is introduced between the other two, to take care of the effect of centrifugal force. The retaining walls and cross walls thus form a series of reinforced-concrete cells or boxes (without top or bottom), placed end to end, and filled with cinders, on which the track is laid.

Cinder filling was selected as being light, porous and cheap. The design is shown clearly by Fig. 1, and by the single-

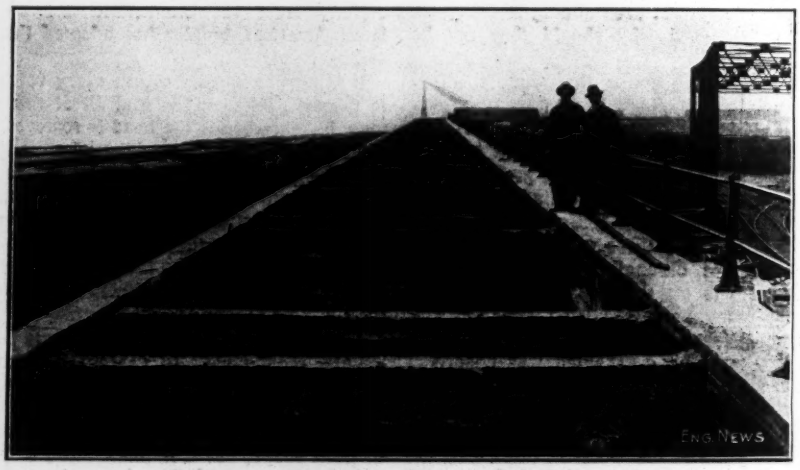

Fig. 1. Viaduct Approach of Cellular Construction; Manufacturers' Ry. Co., St. Louis, Mo.

track cross-section of box-type construction in Fig. 4.*

The retaining walls are only 13½ in. thick at the top, with a batter on one side of ½ in. per ft. The thickness, batter and unit stresses were so chosen as to make the spacing of the horizontal reinforcing bars uniform from top to bottom of the wall. The reinforcement is shown in Fig. 2. In each cross wall, tie rods are embedded, extending to near the outer face of the retaining walls and holding these walls together. The rods are provided with nuts at each end, and pass through vertical 4x4-in. angles which serve to anchor the rods into the retaining walls.

The original design did not contemplate the cross walls, but provided for covering the rods on each side with expanded metal, and plastering this to form a thin partition. This was to prevent the cinder filling from packing on the rods and bending them. To protect the rods from corrosion, they were to be encased in gas pipes, and the space between the rods and the pipes filled with cement grout. Diagonal rods were also to be used at each partition to prevent the side walls from getting out of line at the top. The contractor, however, offered to construct 8-in. concrete cross walls at the same price as the plastered partition, which offer was accepted. As the gas-pipe rod casings had been ordered and delivered, they were used as originally intended, although the fact that the rods were built into the concrete walls made their use unnecessary. The diagonal rods were omitted.

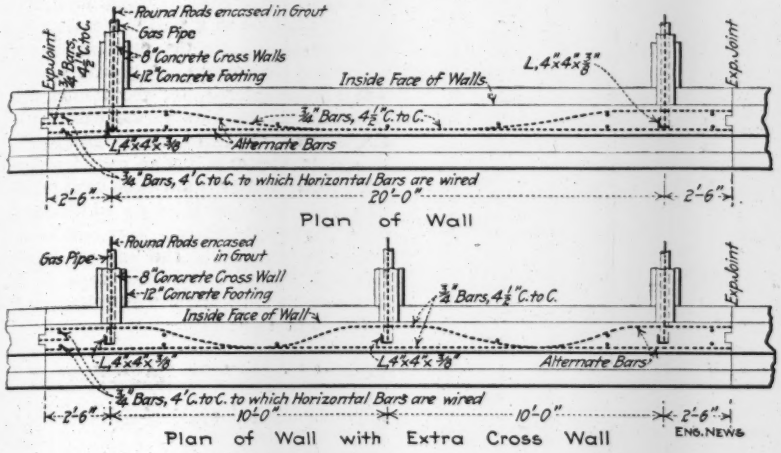

FIG. 2. PLAN OF SIDE WALLS OF VIADUCT APPROACH, SHOWING STEEL REINFORCEMENT

A sidewalk and hand rail were provided on top of one wall, and the exposed faces of the walls were paneled for the sake of appearance. Three subways about 300 ft. apart were provided through the north approach, for communication across the structure. These are rectangular openings 3¼x8¼ ft.

CALCULATIONS FOR APPROACHES—The side walls were designed as horizontal beams, to resist the pressures due to the fill and the superimposed load. The live load was reduced to a square-foot basis, assuming the axle loads to be distributed by the track and ballast over an area equal to the distance between axles multiplied by the clear width between walls. This load per sq.ft. was found to be equivalent to that which would be produced by a fill of cinders 12 ft. deep. The weight of the packed cinders was taken at 60 lb. per cu.ft. The horizontal pressure at the bottom of the wall was then computed, taking the depth of the fill as equal to the depth from ground to top of wall plus the 12 ft. representing the live load.

The horizontal pressure at the top of the wall was assumed to be caused by the 12 ft. of fill just mentioned. While this last assumption is theoretically incorrect, it was made in order to provide for the pressure that would be brought on the wall near the top, by the compacting of the fill under the live load. The horizontal pressure was assumed to increase uniformly from the top to the bottom of the wall. The horizontal forces were computed according to Rankine's theory of earth pressures.

At the top of the side walls, the unit stresses used in designing were made lower than those at the bottom of the wall, to allow for impact. The maximum allowable tension on reinforcing steel of high elastic limit was taken at 12,500 lb. per sq.in. at the top and 20,000 lb. at the bottom of the wall. For concrete, the maximum compression was taken at 500 lb. and 800 lb. per sq.in., respectively, at top and bottom. It was assumed that there would be no impact at the bottom of the wall. The transverse tie-rods (of medium steel) were designed with a unit stress of 13,000 lb. per sq.in., and allowance for impact being made the top of the wall.

COST ESTIMATES FOR APPROACHES—In order to determine the proper point at which to stop the retaining walls, and begin the steel viaduct, the cost per lin.ft. of this "box type" construction was estimated for various heights, and compared with that of a single-track steel viaduct including foundations and timber deck. The figures thus obtained were later embodied in the accompanying diagrams.

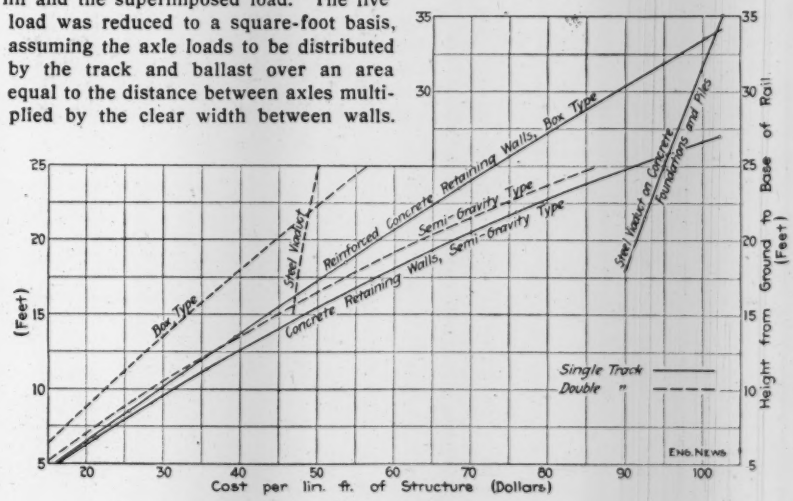

FIG. 3. DIAGRAM OF COMPARATIVE COSTS OF THREE TYPES OF VIADUCT CONSTRUCTION
(For single and double tracks.)

On the first of these, Fig. 3, have been plotted the estimated costs per lin.ft. for

*A somewhat similar construction, but with the cells closed at the bottom, has been used recently in connection with the track elevation of the L. S. & M. S. Ry., at South Chicago, Ill. An inclined track (1% grade) parallel with the elevated tracks and connecting them with other tracks at a lower level is carried just outside the retaining wall of the track elevation, and the fill for this track is held by a second retaining wall, whose top is on the 1% grade. The width over both walls is about 20 ft. The main and outer retaining walls are both built on a continuous foundation slab, and these two parallel walls are connected by transverse walls 7½ to 10 ft. apart. The base, longitudinal walls and cross-walls are all of concrete, and form a cellular construction.—Editor.

three types of single-track and double-track structures: (1), the box type of retaining walls; (2) a semi-gravity type of retaining wall in which the footing course only is reinforced, the rods extending up the back of the wall only far enough to take the tension at the junction between the body of the wall and the footing course; (3) the steel viaduct. Cross-sections of these structures are shown in Fig. 4. It will be seen that for a single-track structure, the box type of retaining wall is much more economical than the semi-gravity type. The box type is more economical than the steel structure up to a height of 22 ft., while the semi-gravity type and the steel structure reach the same cost at a height of only 16 ft. For double-track, the cost curves for the box-type and the steel viaduct intersect at a height of 34 ft., while those of the semi-gravity type and the steel structure intersect at a height of 27 ft.

The second diagram, Fig. 5, was prepared in order to show the variation in cost of the two types of retaining walls for various widths out to out. It will be seen that as the width of the "box type" increases the cost of the cross walls and tie-rods becomes a very considerable part of the total cost; while the only additional cost for the semi-gravity type is that of the extra fill required. Thus for each height, there is a certain width below which the "box type" is the more economical, and above which the advantage lies with the semi-gravity type. The intersection of the cost curves of these two types gives these critical widths for heights of 5 ft. to 35 ft.

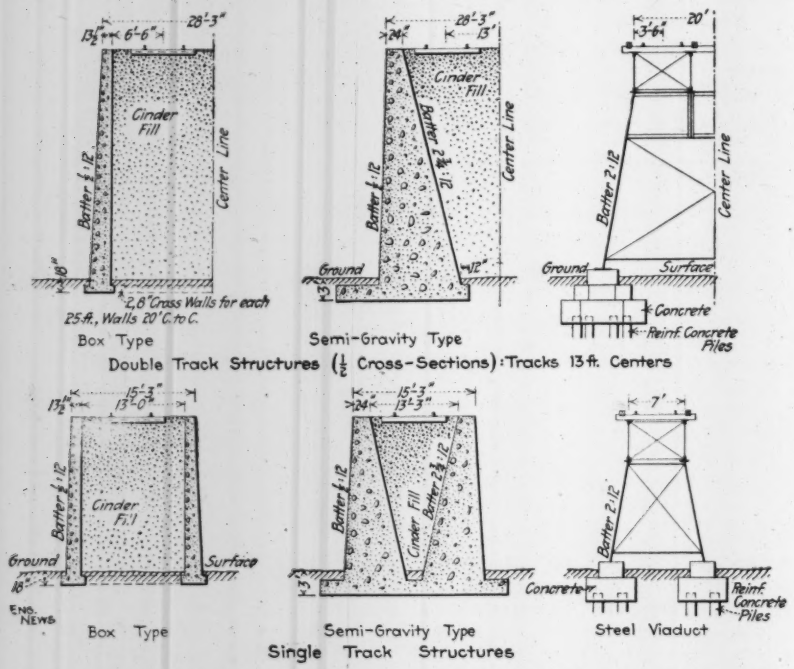

FIG. 4. CROSS-SECTIONS OF TYPES OF CONSTRUCTION WHOSE COMPARATIVE COSTS ARE GIVEN IN FIG. 3

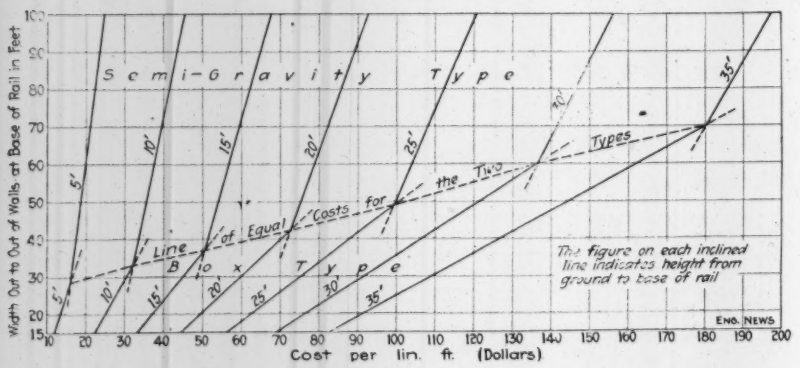
Semi-Gravity Type
Box Type
Width Out to Out of Walls at Base of Rail in Feet
Cost per lin. ft. (Dollars)
Line B
Costs for Equal Heights
The Two Types
The figure on each inclined line indicates height from ground to base of rail
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FIG. 5. DIAGRAM OF COMPARATIVE COSTS OF TWO TYPES OF CONCRETE VIADUCT APPROACHES FOR VARYING HEIGHTS

As there is no overturning moment to be resisted by the walls of the "box type" the footings of these walls were not set as deep in the ground as would be necessary in the case of a wall that resists the earth pressures by the reaction of the soil under the footing. A gravity section was not considered, for the reason that it would be too wide at the base for a single-track structure of the height required in the case under consideration. For greater widths it might be used to advantage. The quantities on which these diagrams were based, were estimated from actual design. The unit prices are based on material and labor costs at St. Louis at the time the structure was built, as given in Table I:

TABLE I. COST OF MATERIALS FOR COMPARATIVE ESTIMATES

Concrete (1:2:4)	$6.00 per cu.yd.
Concrete (1:2½:5)	5.50 per cu.yd.
Excavation	0.35 per cu.yd.
Cinder fill	0.40 per cu.yd.
R. con. piles	1.25 per lin.ft.
Ties	1.00 each
Forms	0.10 per sq.ft.
Ties and guard rails*	0.05 per ft. B. M.
Structural steel	3.75c. per lb.
Rods and angles	3c. per lb.
Reinforcing steel	2.50c. per lb.

*Track rails are not included, as they are common to all types.

The total amount of structural steel was 720 tons, the total cost of which, erected and painted, was $54,420, or 3.78c. per lb. The contract for the retaining walls and foundations was awarded on a unit-price basis; the final estimate on which the contractor was paid being as given in Table II (the unit price for concrete included forms and excavation). The grand total of $110,900 is for the structure complete from end to end of retaining walls, but exclusive of the earth embankments leading up to these walls. These embankments were put in by company forces.

TABLE II. COST OF STRUCTURE AS BUILT

Plain concrete, 302 cu.yd. at $5.66	$1,709
Reinforced concrete, 3210 cu.yd. at $7.75	24,877
Rock excavation, 52 cu.yd. at $2	104
Cinder fill, 8040 cu.yd. at 25c.	2,010
Concrete piling, 4590 lin.ft. at $1.37	8,288
2½-in. gas pipe, 13,457 lin.ft. at 18.5c.	2,489
Track work, 1300 lin.ft. at $1	1,300
Reinforcing steel, 309,000 lb. at 2.37c.	7,323
Rods, angles, etc., 105,065 lb. at 2.9c.	3,046
Filling column bases (lump sum)	108
Total for retaining walls and foundations	$49,256
Total for structural steel	54,420
Track work materials and labor not included above	7,224
Grand total	$110,900

CONSTRUCTION METHODS—The contractor's concrete plant consisted of a mixer supported on a timber tower which was mounted on a railway flat car. The mixer was placed at such an elevation that the concrete could be wheeled from it and dumped into any part of the wall, without using long inclined runways. There were

railway yard tracks parallel with and close to the retaining walls, on each side of the structure. This made it possible to readily move the concrete plant along to any part of the work. Concrete materials were received in cars and fed to the mixer direct, without unloading, storing and re-handling. The concrete materials were raised from the level of the car to that of the mixer by a skip operated by a gasoline engine on the deck of the car. The same engine also ran the mixer.

The footing courses of the walls were laid for some distance before the upper work was started. The 4x4-in. angles and the tie-rods were then set up and braced; after which the forms and reinforcing bars were placed in position and the concreting done. Sufficient forms were provided to take care of three 25-ft. sections of wall. These forms were carefully built of matched material, and were oiled to help in securing a good surface finish. They were altered from time to time to allow for the varying heights of the walls, and were handled by means of a boom on the tower of the portable concrete plant.

This box type of construction was devised by the consulting engineer of the work, Edward Flad, under whose supervision the work was designed and executed. The firm of Brenneke & Fay, of St. Louis, was associated with Mr. Flad in this work, and the writer (as principal assistant engineer for that firm) had charge of the designing of the structure, and also supervised the field engineering methods. Mr. Flad's assistant, C. M. Daily, had charge of the work in the field. The contract for the retaining walls and foundations was awarded to F. C. Mueller & Bro., of St. Louis, who sublet the contract for furnishing and driving the reinforced-concrete piles to the McCully Construction Co., of St. Louis. The Cambria Steel Co. furnished the structural steel for the viaduct, which was erected by the McKeesport Steel Construction Co., of McKeesport, Penn.

The Early History of the Austin (Penn.) Dam

Though the disastrous failure of the Bayless dam at Austin, Penn., on Sept. 30, 1911,* is now a matter of history, there may still be some interest in two photographs of the dam's earlier days, which were found just recently. One of them is a view of the dam under construction; the other shows strikingly its deflected condition after the first filling of the reservoir, in January, 1910.† They are reproduced herewith as Figs. 1 and 2.

It needs only the most general perception of principles of stability and strength of structures to see in Fig. 2 a graphic portrayal of failure. The point of greatest movement is identical with the probable locality of initial failure in the final collapse of the dam, as brought out in our description of the collapsed structure.

The Austin dam may come into future notice, if the efforts made during the past winter to reëstablish the business of the former paper and pulp plant are carried to completion. After the probability of damage suits against the company was fully apparent, the concern went into friendly bankruptcy and its property was sold under a mortgage. A new company was organized, under slightly different name, and the entire plant is to be rebuilt. Work on this has progressed actively. The present status of the work is reported by the Williamsport (Penn.) *Bulletin*, of May 8, 1912, as follows:

> Austin, May 7—The equipment of the Nelson-Beers Co., contractors, which has been in use on the reconstruction of the great mills at Austin, by the Bayless Pulp & Paper Co., of this city, has been shipped back to Binghamton, and the company has virtually wound up its work in this village. The equipment consists of engines, mixers, concrete material, etc.
>
> At the offices of the Bayless company it was stated that the great mills probably will be reopened within the next 60 days, with 200 men laboring in the mills and about 200 more in the woods. This is about the size of the force employed before the business was wiped out by the breaking of the dam. The new mills are built with a view to future increase, if desired, and it may be that the increasing business will require early increase of capacity.

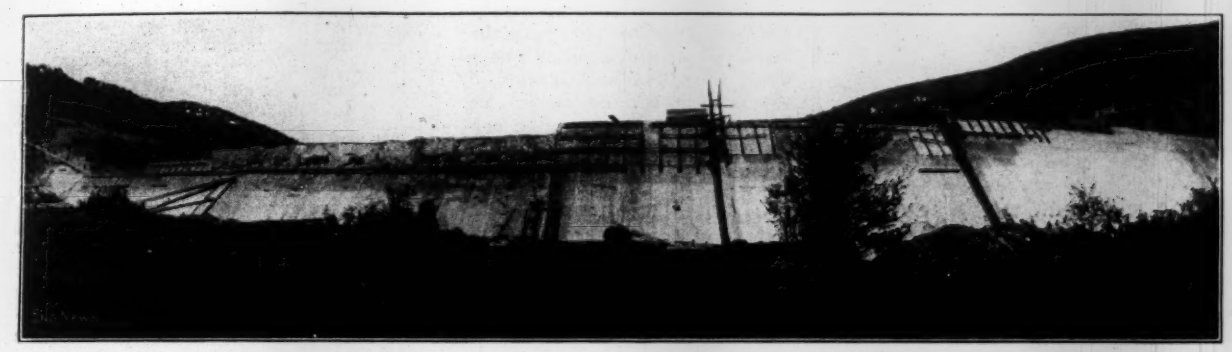

FIG. 1. THE BAYLESS DAM AT AUSTIN, PENN., DURING ITS CONSTRUCTION IN 1910, SEEN FROM DOWNSTREAM

(The concrete ridges and buttresses on the face of the dam in the right-hand portion are the boundaries of the spillway. Just to the right of the spillway was the central point of the first failure or deflection, and at the same point the subsequent complete failure took place.)

FIG. 2. THE BAYLESS DAM AS DEFLECTED AFTER THE FIRST FILLING OF THE RESERVOIR

*Eng. News, Oct. 5, 1911, pp. 417-422.

†Eng. News, Mar. 17, 1910, showed sketches of this.

Technical Education has been the most powerful influence in promoting engineering to its rightful place among the professions. Such at least is the opinion expressed by Edward Hooper in his presidential address before the British Institution of Mining and Metallurgy. He went on to say that the large majority of thinking men who have studied the subject from a disinterested point of view are agreed that thorough theoretical training is essential to the success of the young engineer of the present day.

A 2¼-Mile Tunnel is proposed to reduce the summit elevation of the Chicago, Milwaukee & Puget Sound Ry. in crossing the Cascade Range, but the construction of the tunnel may not be carried out for several years. It would also effect some reduction in the length of the line over the range. To expedite the original completion of the line it was located and constructed with a summit in an open pass, but with a view to the eventual construction of a tunnel at a lower elevation. It has been stated that work would be commenced shortly, but we are informed that this is a mistake. Some preliminary investigation is being made, but the construction is an indefinite matter for future consideration when traffic conditions will warrant the expenditure.

A Proposed Study of Concrete Sands

Defective Concrete Sands

By John R. Freeman*

The first communication below, from John R. Freeman, Consulting Engineer, calls attention to the peculiar behavior of certain kinds of sand, which, while passing all ocular and manual tests of fitness for concrete aggregate, fail to make a concrete of any appreciable strength. Mr. Freeman's request for a chemical or physical explanation of these phenomena is answered by E. E. Free, of the U. S. Bureau of Soils, and R. H. Gaines, of the New York City Board of Water Supply. These explanations admittedly are only tentative, and the true reasons for the condition must remain elusive until a thorough scientific investigation of the subject can be made. Mr. Freeman proposes a public subscription to finance such an investigation. Comment on the question appears on the editorial pages of this issue.

It seems strange that information should still be so meager regarding the behavior of a building material in common use for nearly a hundred years and which in its importance to the civil engineer now outranks all manufactured products except steel.

It is a fact beyond all doubt or question that here and there are banks of sand and gravel which *upon ordinary inspection appear ideally perfect* for making mortar and concrete, but which are nevertheless *dangerous in the extreme* and all because of some ultra-microscopic content probably similar to tannic acid (in a colloidal film around the sand grain), which works in some mysterious way to prevent the union between the cement and the sand grains. I leave the name and the theory of action to some physical chemist.

From what follows, I may here foreshadow the following conclusions:

(1) That a given untried deposit of sand should never be relied upon or used in important work, until after at least a seven-day test with the particular brands of cement which are to be used;

(2) That some one well known brand of cement may give success with this peculiar sand where half a dozen other equally good brands fail utterly;

(3) That this mysterious defect in the sand possibly or probably can be cured by the addition of some exceedingly minute quantity or form of alkali (or anti-colloid) to the cement, which alkali or unknown "x," is so inconspicuous by reason of its composition or quantity that the ordinary commercial chemical analysis of cement does not detect it.

A year or more ago I prepared designs for a dam (not yet built) near the outlet of a small valley about six miles northerly from Providence, R. I., that it was proposed to flood to a height of some 30 ft., in order to provide an additional supply of wash water for a large industrial plant.

In course of the preliminary investigations, numerous test pits were dug in an open level hay-field, near the damsite, to a depth of about 5 ft., which showed up abundant quantities of good looking gravel, with pebbles ranging from 3 in. to ¼ in. in diameter, and thence grading downward into sand grains of various sizes. Examined in the pit or in the shovel or in the hand by rubbing the grains between the fingers for detecting "dirt," this gravel and sand looked all right and very much the same as gravel from which I had recently seen excellent concrete being made on two large structures, and I believed I had found a veritable bonanza of low-cost concrete aggregate suitable for dam-building purposes.

But, being a great believer in the mechanical analysis, screening and regrading of sands and gravels as a means toward obtaining the best combination of imperviousness, strength and economy in concrete, I had representative samples aggregrating perhaps 150 lb. taken from three different pits at levels of from 2 to 5 ft. beneath the surface as I now recall them, and sent to Sanford E. Thompson for mechanical analysis and tests, with instructions to promptly report the seven-day test, without waiting for the 28-day tests which were also in process, in order that specifications and estimates might be hurried along.

He had trouble in making his report on schedule time, because at the end of seven days some of the samples of 1:3 mix—six days in water—were hardly more than strong enough to retain their integrity while being placed in the clamps, prior to applying any load. The cement used was a brand that he had been using in his laboratory for experiments on other sands and gravels and which had uniformly given good results. It is noted in this article as brand "B."

An abstract of one series of the tests with sands screened from three of the pits is as shown in Table I.

It will be noted that the specimens of the 1:1 mix at the end of two days' storage in moist air showed fairly good strength, and that the briquettes that had been stored under water, upon removal therefrom to the dry air hardened somewhat rapidly. In another set of briquettes that was removed from the water four days after mixing and then partially dried in air (see Sample 3) tested higher than Sample 4; and it was noted that the surface of these partially dried briquettes was very much harder than the interior.

The important fact shown by these tests was that most of the 1:3 wet specimens gave only one-fifth part of the normal strength at seven days.

Mr. Thompson thought this peculiar action might possibly be in part due to a temporary muddiness in the Newton City water, or that possibly the cement was affected by the extreme heat prevailing at the time of the test, and therefore started a new series of tests, using distilled water and storing the new briquettes, some in moist air, some in dry air, and some in water, pending the tests. But with these also the strength was defective at the end of seven days, being found not over 40 or 50 lb. per sq.in., whereas samples made from the same cement at the same time with standard sand all tested at 200 lb. per sq.in. or over.

Further samples of the sand and gravel were then obtained and after giving them a thorough washing, 1:3 mixtures with cement and tests of tensile strength were made. These samples from washed sand gave tolerably good results, as shown in Table II.

The important lesson of this series of tests was that washing the sand caused it

TABLE I

Sample	Brand of cement	Proportions	Storage in Days, In moist Air	In water	In dry Air	Age	Strength in lb. per sq.in. Standard Ottawa Sand, Tension	Standard Ottawa Sand, Compression	Sand from No. 1 Gravel Pit, Tension	Sand from No. 1 Gravel Pit, Compression	Sand from No. 2 Gravel Pit, Tension	Sand from No. 2 Gravel Pit, Compression	Sand from No. 3 Gravel Pit, Tension	Sand from No. 3 Gravel Pit, Compression
1	A.....	1 : 1	2			2	283	.....	176	.....	180	.....	296	.....
2	A.....	1 : 1	2		11	13	.....	5,500	.....	5,430	.....	4,150	.....	5,350
3	B.....	1 : 3	1	3	3	7	235	.....	75	.....	40	.....	67	.....
4	B.....	1 : 3	1	6		7	210	.....	42	.....	35	.....	38	.....
5	A.....	1 : 3	1	6		7	242	.....	120	.....	38	.....	175	.....
6	A.....	1 : 3	1	6	5	12	.....	2,760	.....	2,900	.....	1,800	.....	3,320
7	B.....	1 : 3	1	6		7	200	.....	50	.....	45	.....	45	.....
	Average..	1 : 3					222	.....	72	.....	39	.....	89	.....

*Compression specimens are the halves of briquettes broken in tension and recorded on preceding line.

*Consulting Engineer, Banigan Building, Providence, R. I.

TABLE II. TESTS OF SECOND LOT OF GRAVELS AND SANDS SENT AUGUST 8, 1910

Tests of gravels made with sand obtained by screening gravel over a sieve with five meshes per linear inch. Age, 7 and 28 days

The Values Are Tensile Strength in Pounds per Square Inch

Brand of cement		Brand "B"*					Brand "A"**
Proportions	1 to 3	1 to 3	1 to 3	1 to 3	1 to 3	1 to 3	1 to 3
	Sand Screened from Gravel						
	Standard Ottawa sand	Sample No. 1	Sample No. 3	Sample No. 4	Sand Sample No. 5	No. 3 gravel	No. 3 gravel
Unwashed sand (7 days) (stored in air)***	246	53	90	140	65	213	30
Unwashed sand**** (7 days)	263	25	43	90	82	163	soft
(28 days)	333	140	150	180	123	233	90
Washed sand (7 days) (stored in air)***		216	230				
Washed sand**** (7 days)		207	217				
(28 days)		240	258				

*A mixture of two parts Sample No. 1 with three parts No. 5, mixed 1 to 3, unwashed, stored seven days in air, was too soft to handle.
**This cement had been in laboratory for some time, which may have influenced the test.
***Only one briquette of each lot was stored in moist air.
****Briquettes stored one day moist closet and remainder of time in water.

to give nearly normal strength in 1:3 mix, at end of seven days. The samples from unwashed sand mixed 1 cement to 3 sand gave substantially the same weak mortar as before in the seven-day tests, notwithstanding these new samples had been taken from a depth 6 ft. below the natural surface, with a view to avoiding possible surface contamination. Two of the specimens made up 1:3 with the unwashed sand from 6 ft. below the surface were too soft to handle at the end of seven days.

Because of this washing having so important an effect upon sand which did not look "dirty," I requested that another sample be washed and the wash water subjected to careful chemical analysis. I was called West about this time and my ideas as to what should be sought in this water analysis were not fully comprehended, but a sample of water that had been used in washing the gravel was turned over to the laboratory of the Massachusetts State Board of Health and analyzed according to the standard methods used for analyses of drinking water, and the report came back showing that this wash-water was high in albuminoid ammonia; but as is well known to water-supply engineers, a return of "albuminoid ammonia" does not mean that this definite substance existed in the sample as collected. On the contrary, this term may be merely a convenient designation of the substance into which the chemist transmutes various organic compounds that existed in the sample, in order that he may weigh this substance and obtain an index of the quantity of undecomposed organic material possibly subject to decay that existed in the sample. This water analysis, therefore, did not give all the information that I had hoped for.

TABLE III. RESULTS OF TESTS

Comparison of Tests of Mortar Made from Sand Screened from No. 1 Gravel, Received Nov. 16, 1910, with that Made from Standard Ottawa Sand

Tensile Strengths in Pounds per Square Inch

Brand of cement	1:3 Standard sand	1:3 No. 1 Sand 1 day in moist air 6 days water	1:3 No. 1 Sand 1 day in moist air 6 days air
D	206	38	80
E	162	0	0
F	235	0	44
G	211	0	27
H	272	0	67
A	238	174	202
C	197	72	132
B	226	13	52
K	204	0	40

Although Mr. Thompson told me that in a previous case he had traced the trouble to organic matter, evidently of vegetable origin, which had apparently washed down over the surface into the sand and gravel below, and had stated that so small a percentage as 2% of "silt" which had been washed out from other poor sand when added to the standard sand had been found to reduce the strength of 1:3 mortar from about 200 lb. to 70 lb., I could not see how in this case sufficient deleterious organic matter or "silt" could have worked down from the surface of an isolated mowing field or potato field through 3 or 4 ft. of good filtering material, so as to permeate all of a large mass of gravel, many acres in extent.

It remained incomprehensible to me how samples of gravel, which from my limited geological attainment I assumed had been lying undisturbed for more than ten thousand years, or since the glacial epoch, buried 2 to 4 ft. beneath the average bottom of the grass roots in a broad, level, cultivated meadow, a hundred feet distant from the bank of the little rivulet, and seldom, if ever, subject to flood overflow, could have become sufficiently contaminated by organic material so that a respectable brand of portland cement would decline to unite with it.

Mr. Thompson made some further experiments in following out the contamination idea, by stirring into the mixture various percentages of "fertilizer," 1, 2 and 3%, respectively, with no important result.

H. W. Clarke, Chief Chemist of the Mass. State Board of Health, in following out the albuminoid ammonia idea, had found that this substance affected the strength in a marked degree; for example, on adding 60 parts per 100,000, which it will be noted is only 0.06% to standard sand, the tensile strength of the 1:3 mix was reduced in a seven-day test from 219 lb. to 35 lb., and in the 28-day test was reduced from 316 lb. to 48 lb., by this introduction of albuminoid ammonia which, as I understand it, was extracted from our defective sand.

In chemical water analysis many different substances come out in the weighing as albuminoid ammonia, some animal, some vegetable, some quickly decomposing like algæ, some permanent like the tannin compounds which color swamp water a deep brown.

Not being a professional chemist, my conception of "albuminoid ammonia" was such that I still failed to understand how it could be in existence, deep underground, in a gravel bed.

I had already perused the report of the informal discussion at the convention of the American Society of Civil Engineers, on July 8, 1909, by Messrs. Thompson, Leslie, Howard, Humphrey, Whinery, Richardson, *et al.*, and I sought additional light from other engineering friends.

On questioning Allen Hazen, he told me that a few years ago when building certain extensive filtration works, he had encountered a similar but less serious trouble with sand, from which he found relief through a series of tests made by using the same given sand with all the various brands of portland cement then found available in the market, and that he found one particular brand (brand "C," herein) gave results that were conspicuously superior to those with all the other brands, and in fact so satisfactory that this brand was used for completing the work.

I thereupon requested Mr. Thompson to run a similar line of tests with all the different brands of cement that were conveniently available.

Mr. Thompson found that brand "A" cement, from one of that company's three mills gave results remarkably different from all the others, giving with this sand about 80% of the strength found with standard sand; while brand "C" cement on the seven-day test gave

TABLE IV

	A† (1)	A† (2)	C	G	E	B
Silica	23.50	23.30	22.62	21.79	23.82	21.63
Alumina	6.96	6.85	6.77	7.52	5.37	8.12
Ferric oxide	2.86	2.71	2.83	2.83	4.09	3.15
Lime	62.37	63.30	63.73	64.15	63.66	62.44
Magnesia*	3.08	3.39	3.80	2.36	1.78	2.92
Sulphuric anhydride	1.44	1.32	1.56	1.51	1.24	1.85
Sodium oxide Na_2O	.....	0.52	.....	.....	0.49	.....
Potassium oxide	.....	0.11	.....	.....	0.08	.....

This chemical analysis calculated to ignition and alkali free basis.
*Based on average tests by two chemists, Sherman and Clark.
†Brand "A(1)" is from different bag of cement that gave results similar to brand "B"; brand "A(2)" gave satisfactory results.

about one-third the normal and several other brands of good repute gave from 20% of normal to practically no strength at all, after one day in moist air plus six days in water in the practical 1:3 mix. See Table III.

Tests with addition of hydrated lime, which also check comparison of brands "B" and "A" and showed that the addition of lime would not cure the trouble with this particular sand, although cases were known where it had been helpful.

	1:3 standard sand			1:3 No. 1 sand	
Hydrated lime added	0%	5%	10%	5%	10%
Brand "A" (see above)	...	257	260	214	205
Brand "B"	253	267	249	45	57

The chemical analysis of these different cements, as per Table IV, gave no sufficient explanation and my puzzled state of mind was not relieved.

A little later I had the good fortune to make the acquaintance of E. E. Free, an investigator in the borderland between chemistry and physics, for some years past connected with the Department of Agriculture at Washington, D. C., and now engaged upon physical and chemical investigations for the Bureau of Soils, with particular reference to the means by which the plant is able to take into solution the mineral ingredients, silica, etc., required to strengthen its constitution.

I saw by the way Mr. Free's face was lighting up before my story was half told that I was in a fair way to receive some valuable information, and he soon told me that he had run across analogous phenomena in studying the relative fertility of different soils. He said that in popular terms the situation could perhaps best be explained by considering that an extremely thin film of a complex form of tannic acid surrounded each grain of the troublesome sand, and prevented its chemical attachment to the cement; that these complex organic acids had a strong tendency to thus spread themselves thinly and strongly adherent over the outside of the siliceous material and that the reason why one particular brand of cement was able to break through this filmy barrier so as to get a firm grip upon the sand grain, was because of it containing a small quantity of some form of alkali that united with this acid film, or somehow changed its colloidal state.

Mr. Free's line of thought gave such promise of being helpful and his ability to describe the situation in terms of physical chemistry is so much better than my own, that I subsequently asked him to write out the substance of his statements, which is presented below.

Since then, in Baltimore, Walter W. Crosby, Chief Engineer of the Maryland Roads Commission, gave me some interesting facts from the other end of the field, in calling attention to the remarkable influence of tannic acid in producing a colloidal condition in clay, and told me of his own experiments in the improvement wrought in roads with the waste liquor from pulp mills which is rich in tannic acids and other organic materials. More recently, William T. Donnelly, of Brooklyn, N. Y., told me of the part played in the fertility of soils by the tannic acid from the accumulation of dead leaves beneath the trees leaching and filtering down into the soil and in some way preparing the mineral for plant food.

I respectfully invoke the aid of ENGINEERING NEWS to direct the thoughts of our students of the chemistry of portland cement toward finding out just what is the matter and toward finding an antidote for tannic acid or albuminoid ammonia or the unknown colloid in certain innocent looking sand and gravel banks and toward explaining the strange fact that a certain brand of cement could make good work when mixed with these peculiar sands, while four out of five other brands failed utterly.

I hope you will invite discussion from all over the country from cement chemists and engineers and "colloidal physicists," upon this line of phenomena.

Let us first collect all the examples that we can of good looking sands that have failed, locate their beds, so th. conditions can be studied in situ and more samples obtained, and so that influence of time can be studied *in situ* and more samples work; for our brief experiments tend to show that time brought a noteworthy increase in strength, and some other tests indicated that prolonged attention in the mixer helped a little.

Possible Explanation of the Defective Sands

BY E. E. FREE*

There can be little question that the failure of certain sands to adhere to cement is due to the presence of organic materials in and on the sand. Whether we imagine this material to exist as thin liquid films about the sand grains or whether we assume it present in so called colloidal condition, is unimportant. Indeed, in the present state of knowledge in this field these two opinions represent hardly more than different terminologies. The real differences, if any, are unknown.

Organic substances of various kinds are known to be commonly present in soils, especially the soils of river bottoms and of swamps, and some few of these substances have been extracted and isolated. Prominent among these is the diozystearic acid, discovered and investigated by Schreiner and his associates. Prominent also, though less clearly known, are the so called *Gelbsaure* bodies similar to tannic and gallic acids, and the complex humus bodies long known to soil investigators as the humic and ulmic acids. These bodies (and many others, known and unknown) exist in soils, and nearly all possess the so called colloidal properties which would cause coatings of them to impart to sand grains the troublesome characteristics observed. Furthermore, these bodies are all soluble or suspensible in water and would be largely extracted from their solution or suspension by sand grains and retained on the surfaces of these grains by virtue of the processes called adsorption. It is quite to be expected that drainage waters from swampy or forest lands would be highly charged with these bodies, and that sand or gravel in beds percolated by such waters would gradually receive surface coatings of them, which coatings would be quite sufficient to account for the observed phenomena. It should be noted also that the ordinary procedure for the chemical analysis of water would detect these bodies, if at all, only as albuminoid nitrogen or as oxygen consumed.

Turning next to a possible remedy for the condition it will be well to preface any suggestion with a brief note of certain properties of these organic bodies, which properties concern materially the matter under discussion. Such organic bodies, like all materials in, or approximating, the colloidal state, are very sensitive to changes in their surroundings (as, e.g., to temperature) and especially to additions of very small quantities of certain foreign substances, especially soluble inorganic salts and mineral acids. These substances produce a so called "flocculation" of the suspended colloidal material, causing it to agglomerate into large aggregates and to settle out. It is to be expected that such flocculating agents would break the films coating the sand particles and restore the sand to useful condition. It should be noted, however, that the action of these flocculating agents is very complex and largely specific. An agent which will produce a flocculation under one set of conditions may have an exactly opposite effect under other conditions, on another colloid, or in solutions of other concentrations.

Returning to the question of a practicable remedy, I would suggest that such a remedy might lie in one or both of two procedures: (1) Removal of the offending material (as by washing or burning), and (2) rendering it innocuous by flocculation. The first of these procedures can be passed over for the present. As to the second, it is obvious that a necessary preliminary to the suggestion of an actual method must be a more exact knowledge of the nature of the material (or materials) and of the flocculating agents which would affect it. This knowledge is not at hand, and I would therefore suggest that the first step in the investigation of this trouble be the nearer analysis of the organic materials which lie at its root.

Such investigations would contemplate

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no small amount of work and would require a very high order of analytical skill, but that they are by no means impossible is amply proved by recent successful investigations into the nature of the organic matter of arable soils. When there shall have been obtained such accurate and fairly detailed knowledge concerning the chemical and physical nature of the organic colloids or coatings, the suggestion of suitable flocculating or destroying agents will be simple, and their testing will be easy.

In advance of the obtaining of such knowledge it is possible to offer very little assistance. I might make a tentative suggestion of the employment of weak nitric acid or of caustic lime. Both of these materials are in general strong flocculating agents and might be found efficacious. If employed, the nitric acid should be added in weak solution to the sand before mixing; the lime should be added to the *dry* cement, also before mixing. It might be, also, that the use for mixing purposes of ordinary salt water (as of the ocean) instead of fresh might be of assistance.

All these suggestions are exceedingly tentative and are offered with some hesitation. The matter concerns the reactions of these added materials not only on the organic matter which it is desired to affect, but also on the process of the setting of the cement itself, and it is nearly impossible to predict just what any added material will do. Furthermore, it is very probable that the troublesome organic materials are not of the same composition in all sands which show the effect, and a procedure which was successful with one sand might utterly fail with another.

I believe that these various matters can be elucidated only by detailed scientific study in the way I have outlined, and I am convinced that the scientific method of approach is in this case also the most practical and the most likely to lead rapidly to useful results. As a means for the prosecution of such an investigation, I might venture to make another suggestion, namely, that the work be undertaken in some university under the industrial fellowship arrangement recently so highly and successfully developed by Prof. R. K. Duncan of the University of Pittsburg. If funds can be provided for such a fellowship, I shall be glad to attempt the arrangements with some satisfactory university, and I am quite sure that there will be no difficulty on that score. I should also be glad to be of any desired assistance in securing a competent man to conduct the investigation, and here also I anticipate little difficulty. As to finances, I should say that the first year's expenses of such a fellowship ought not to exceed $2000 for salary, and $500 or $1000 additional for special apparatus and supplies. The work might so develop that additional funds would be advisable later, but such increases could be faced best when their desirability became evident. As to practical (that is, financial) returns, I can say only that they seem to me to be very probable. It is impossible to guarantee the outcome of any scientific research, but this particular investigation offers a better hope of valuable and almost immediate returns than any problem which has come to my attention. The experimentation offers no insuperable difficulties and the value of a successful result is sufficiently obvious.

Further Thoughts on the Physical Chemistry of the Defective Sands

By Richard H. Gaines*

The term "sand" is vaguely employed to denote very different substances. Even if we restrict the word to the granular product of rock decomposition it will be found that we have many dissimilar bodies to deal with. Sands differ not only in chemical composition, but in physical constitution.

Recent experience has shown that data which ordinarily determine the choice of sands for use in concrete, the granulometric analysis, the amount of voids, and the mineral constituents afford no positive basis for predicting what the tensile strength will be. Every engineer has had acquaintance with mortars that delayed in hardening for unusual periods without apparent reason. Others than Mr. Freeman have encountered sands that looked all right, and gave a satisfactory mechanical analysis, but which produced worthless mortar, and in some cases caused actual failure of the structures. Investigation has usually traced the trouble to some small quantity of organic matter which was believed to have acted either chemically or physically in effecting the strength of the mortar, especially in the early stages. This result is reported to have been produced by not more than 0.1 to 0.2% of organic matter (*Proceedings* American Society for Testing Materials, Vol. XI, p. 522).

A sand may contain detrimental impurities such as humus, slime or other organic material scarcely observable to the naked eye or possible of detection by ordinary chemical methods. It goes without saying that all these substances should be eliminated or destroyed as completely as possible, which can usually be accomplished by heat or by washing the sands.

While all will agree as to the necessity of careful examination before approval of any material for construction, we must not over-rate the importance of the nature of sand. The choice of sand is generally determined by the condition of use. Unless in work exposed to sea water, or alkaline water, or for some special purpose, even with mediocre sands sufficiently satisfactory results will be obtained.

The inference from data collected among engineers relating to sands which give rise to the extraordinary results in mortar described by Mr. Freeman is that the occurrence is somewhat rare. The strength of a large number of different sands was studied in 1907-8 by Humphrey and Jordan of the United States Geological Survey. The sands were collected from nearly every variety of deposit including river and bank sands, glacial and alluvial deposits from different localities of the six middle Western states—Ohio, Indiana, Michigan, Illinois, Missouri and Kansas. All these sands were mixed with standard brands of portland cement and subjected to thorough test. The results showed that while there were marked differences in early tensile, compressive and transverse strengths, the uniformity was much greater after six months. In these experiments no instance was reported of a sand that failed to give a fairly good mortar.

Mr. Freeman asks a theory to account for the failure to obtain satisfactory mortar with sand like that found near Providence—or the physico-chemical process by which a seemingly negligible quantity of organic matter may retard or prevent the union of cement and sand grains. The problem presented offers an enticing opportunity to the engineer who likes to soar in the realm of theory. The most precise knowledge of the *action mecanique* here desired might not carry one very far in settling a practical question. However, two explanations based on what we know of chemical kinetics and molecular physics are suggested. Partial or complete inhibition of the progress of reactions of the character under discussion may occur from two different causes:

1. Catalytic action, due to the presence of a negative catalyser.

2. Sterical hindrance, arising from adsorption phenomena.

Negative Catalysis—It has long been known that certain substances promote or retard the course of certain chemical reactions without any change taking place in the substances themselves. The more familiar form of catalysis is in acceleration of the velocity of reaction. The reverse phenomenon has also been frequently observed, where the mere presence of a small quanity of a given substance has immensely retarded the progress of chemical reactions. Many examples of negative catalysis might be given, but the following will suffice as an illustration. Bigelow (*Zeitschr. physik. Chem.* 10, 30) showed that the velocity of oxidation of sodium sulphite by

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oxygen is greatly diminished by minute additions of organic substances, such as mannite and benzyl alcohol, and even that it is practically stopped. The retardation is so enormously great that it is strongly noticeable with solutions of mannite of the concentration of $\frac{1}{10000}$ normal. Catalytic phenomena are met with on every hand both in nature and in the laboratory, and it is therefore not unlikely that in many cases of slowly hardening mortars minute amounts of foreign substances behave as negative catalysers as above described and interpose effective resistance to the reactions desired.

It is now known that certain organic films on sand even of molecular thickness may greatly retard the ordinary reactions in mortar. Obviously a suprachemical analysis would be needed to ascertain the true character of such minute quantity of material. The effect produced by these interfacial films might be termed micro-heterogeneous catalyses.

STERIC HINDRANCE—Another explanation deduced from theoretical considerations, but not easy of experimental verification, is that the peculiar action of the sands is due adsorption phenomena. The idea here is an extension of Nernst's generalization on the velocity of reaction in a heterogeneous system of several phases. An important expansion of physical chemistry has lately been made in a direction in which considerations of space play a determining part. The velocity of reaction in such a heterogeneous system as a mixture of cement, sand and water is largely dependent upon the extent and nature of the separating surface between the reacting phases, and also upon other circumstances of a secondary nature, such as the diffusion capacity. Even a very small quantity of colloidal albuminous matter of the character reported in the wash water of the sand would alter the equilibrium relations and might indefinitely retard the ordinary reactions in mortar. Complex substances which form colloidal solutions have the capacity of adsorbing other substances from these solutions, which accumulating on the surface of their particles, interfere sterically with their chemical reactions. For this reason, such colloidal substances do not in their reactions obey the ordinary laws of mass action. In the phenomenon just described the assumption is made that in any molecule each atom vibrates about a certain center, and that the amplitude of this vibration is not negligible in comparison with the size of the molecule itself. It is obvious that if this be granted the atom will require for its motion a space in excess of its own volume. Now if we suppose that another atom forces its way into the vibration sphere of the first atom, collisions between the two will occur and it is probable that one or another will be spatially impeded or hindered from entering into the process. When several compounds react together, groups are generally formed. Obviously the groups will not react with one another until they come within the range of each other's attractions, and if we can prevent their doing so we can hinder the progress of the reaction. But this we should be able to do by interposing the resistance offered by the atoms of certain foreign substances. Thus the action may be impeded or stopped, which has been demonstrated to be actually the case in a number of processes. The failure of various chemical reactions to take place in the presence of hydrolysis products of protein (impurities of the same general character as found in the sand) has not yet been explained on any other hypothesis.

Whatever may have been the physical explanation of the trouble with Mr. Freeman's sand, the practical remedy is the complete elimination of the offending organic substance by heat or washing.

Maintenance of High-Pressure Fire Service Systems*

An auxiliary high-pressure water system for fire protection is accepted as an important part of fire-fighting machinery and an absolute necessity for the safeguarding of property in highly congested districts in the larger cities. With the above accepted we are no longer interested in the type of the pumping equipment or pipe for the distribution system, which become matters of detail, subject to local conditions, and which we can well trust to the engineers who are in charge of the design and construction of the plant. Our interest now takes in the question of the limitations of the high-pressure systems, their maintenance and operation. These are well worthy of study, as we all know how easy it is to build up something new, but how much more difficult and less interesting is the work of maintaining it in its original condition.

The records of systems completed or in course of construction show that the installations are rapidly growing, particularly in the larger cities. The mileage varies from a maximum for the Borough of Manhattan of New York City of 94 miles in service (with additions of 14 miles under contract and proposed extensions of 64 miles for the next three years) to the smaller installations of 10,000 ft., into which water is pumped from fireboats.

PROTECTION AGAINST WEAK POINTS ON THE SYSTEM

Practically all of the larger systems have merely nominal pressure maintained in the mains, with the pressure applied from specially constructed pumping stations on receipt of alarm. San Francisco is an exception to this, as pressures ranging from 80 to 140 lb. will normally be maintained (with the possibility of increase if necessary); and New York, on the completion of the Catskill aqueduct, will be able to supply approximately 100 lb. constantly on the mains. That means that for practically every fire or alarm of fire in the district covered, the pressure is raised over many miles of mains, often exposed in excavations in the street with the consequent danger of blowing out joints, breaking or stretching of tie rods and many other sources of accident. This source of weakness can be remedied and such accidents provided against in three ways:

1. By periodical pressure tests, say twice a day, with the pressure raised to 200 lb. and held there for some time. This will tend to show up the weak points at a time when a force is available for prompt repair and not when the pressure is being raised for a fire.

2. By separation of the district protected into two or more divisions, where such division is possible, with all connecting valves normally closed. These valves are opened when a large fire renders it necessary, by men assigned to the work. This reduces the number of times each section is under pressure in proportion to the number of subdivisions into which the distribution system is divided. The earlier sections of the New York system are about to be so subdivided.

3. By a complete installation on the duplex plan, in which independent systems, capable of delivering the desired amount of water at the desired pressure, are placed in alternate streets and joined at two or more points on the larger mains through electrically operated valves controlled at the pumping station. With this system the danger of a complete breakdown is most remote, as should an accident happen to one part of the system it would show at once on the meters at the pumping station; and by throwing the switch controlling the operation of the motor-operated valves separating the two systems, the injured part would be cut out and pumping would be continued into that part which is intact. The last two extensions made to the New York system and the proposed installation at Boston are of this type. This plan adds about 10% to the original cost of the distribution system.

No municipality in which an auxiliary high-pressure system has been installed should consider that its work is complete by thus covering the territory of the maximum hazard. Every effort should be made to modernize and maintain at its highest efficiency the regular fire-department force and portable apparatus, so that should there be a sudden failure of the high-pressure system, due to break-

*Abstract of a report presented at the annual convention of the National Fire Protection Association, held at Chicago, May 14 to 16, by the Committee on High-Pressure Fire Service Systems (H. B. Machen, New York, Chairman).

age in a main, there would be available at instant notice sufficient of the regular equipment to properly carry on the work of the firemen pending the restoration of pressure to the high-pressure mains. As an illustration may be cited the fire at Elizabeth and Grand Sts., New York, in the early part of December, 1908, less than six months after the system had been placed in regular service.

Shortly after the pressure had been raised, the firemen at the fire and the engineman at the pumping station were horrified to see the pressure drop to practically zero, although additional pumps were started in an endeavor to hold the pressure at the 125 lb. called for. A few moments' pumping showed that a large main had broken somewhere in the system, with all the water evidently going out through the break and not through the hydrants, so the inlets to the system were closed and it was completely out of service. The regular fire engines were immediately called into service to cope with the fire, and the leak or break sought for. After nearly an hour's search, a night watchman reported a large quantity of water in a subway excavation near Broome and Centre Sts., and further investigation showed that there had been a break in a 20-in. main at this location. The broken section was immediately shut out and the system restored into service. The trouble was caused by insufficient bracing of the main where it crossed a subway excavation, a piece of work over which neither the water nor fire departments had any control, and as all the excavation was planked over to carry traffic, no sign of the break showed on the street surface. The hour being early in the morning, no men were working, so there was no one to give notice of the trouble.

This condition of suspended and inadequately braced mains is not an isolated one, but one of frequent occurrence, due to the constant digging up of our streets for various purposes. In the particular instance above noted, while it occurred within three city blocks of the fire, the firemen did not know the source of the trouble nor how near it was; while the Water Department force, ready to immediately proceed to the break and make the necessary shutdown of the broken section, stood idly in the repair company headquarters, waiting for a report as to where (on the 52 miles then in service) they should proceed. It should be noted, however, that the case cited is the only one in which the New York high-pressure system has failed in service from any cause.

Maintenance and Operation

Pumping Stations—No effort should be spared to employ only the best grade of men available for the operation of the stations and to have them kept up to the highest efficiency, both by constant supervision of the stations and by so laying out the routine work that no part of the equipment can be overlooked. Thus, should the system not be called into service during any 12-hour period, it is advisable to run a 20- to 30-minute test at slightly above usual working pressure. During the test the pumps should each be operated in turn, giving each pump and motor a chance to be examined under service conditions. This also places the mains, valves and hydrants under pressure, and the test can be arranged to take place at such a time as the repair force is at its headquarters ready to make immediate repair of any defect developed.

The motor-operated valves should be operated daily, being opened and closed completely before returning to their normal position in order to test the signal lights which indicate whether the valve is open or closed. Pressure gages, both on suction and discharge, should be of the recording type, as should be the Venturi meters indicating the quantity of water pumped, and the fire alarm, which should be recorded on the station blotter as well as by a recording instrument.

Distribution System—For the mains, a hose and meter-testing force should be constantly working over the system, closing out individual blocks and feeding water across the closure through a meter and hose by connecting a hydrant outside with a hydrant inside the closed section. Should any large quantity appear to be leaking away, the leak should be located and repaired immediately. Should the leakage be small, the amount need simply be recorded for future reference. A regular patrol should also be provided to cover the entire district, to safeguard against excavations being made in too close proximity to the mains.

An individual record of each valve should be kept, beginning with its date of installation. For this purpose each valve should be given an individual number. The placing of a number plate in the valve vault is convenient and assures accuracy as to location. No valve should be touched without a proper report being forwarded covering the work done. The valve vault covers should be of a design different from the regular water-department cover, so that by no mistake would the valves be operated except by the repair company specially assigned to the high-pressure work.

In order to keep the hydrants at all times in proper condition, an inspection should be made of each hydrant immediately after it has been in service at a fire. The Fire Department should therefore immediately notify the repair company or the official responsible for the condition of the high-pressure system of each hydrant that has been in service. On receipt of the notice a competent man should be dispatched to make the necessary examination. He should be capable of making at once all minor repairs and should report that he had examined the hydrant and left it in good condition. This fixes definitely the responsibility for the condition of the hydrant and makes the men most careful. Should extensive repairs be needed, the man should attach some sign to the hydrant indicating that it is out of service (to warn a fire company from attempting to operate it before repair has been made), and then notify the nearest fire company verbally that the hydrant is out of service for repairs. Fire-department headquarters to be similarly notified in writing, through the main office. On the completion of the repairs, the sign should be removed and the Fire Department notified as before.

Independent of these special examinations, periodic and regular annual examinations of all hydrants and valves should be made, in which all moving parts are carefully cleaned, oiled and operated.

All of the above routine should be a matter of record in the supervising engineer's office, so that at any time the condition of each hydrant or valve can be known—when it was used and when and how repaired, with the name of the man who last reported it "in good condition." Then, on a properly mounted wall map, by means of pins with colored heads, each valve or hydrant closed down or out of service is shown, thus keeping constantly before the eye of the supervisor a record of that part of the system out of service. Should the above outlined procedure be carried out with such changes as local conditions may require, we can feel assured that our high-pressure systems will respond at all times to all legitimate demands for both pressure and volume of water.

An Automatic Fire Alarm recently developed for installation in connection with the Grinnell sprinkler system consists of a device whereby the water flowing through a sprinkler is made to put in operation a gong on the outside of the building. A valve is installed in the piping system in the riser traversed by all water flowing through the sprinklers. When the flow starts, a rubber-faced clapper is raised from its bronze seat and permits water to flow to a water motor mounted on the outside wall of the building. The motor consists of a small bucket-wheel in a metal casing, and is mounted on the inside of the wall in order to avoid freezing. The shaft of the motor extends through the wall and carries on its outer end a striker which is rotated by the motor shaft and rings a large gong on the outside of the wall. The striker and gong are inclosed in a galvanized-iron hood to protect them from obstruction by snow, birds' nests, etc. An alternate arrangement is to have the alarm valve permit water to flow into a magnetic circuit closer so as to start the ringing of an electric gong. The device is being put on the market by the General Fire Extinguisher Co. under the name of the straightway alarm valve.

International Navigation Congress Papers

Navigation of Large but Shallow Rivers

By Lansing H. Beach*

Four papers have been presented on this subject as follows:

No. 47. A comparison of the relative economies of side-wheel and propeller-in-tunnel towboats, by Herr Director R. Blümcke, of Mannheim.

No. 48. Reasons for the decadence of traffic on the Mississippi River, by C. McD. Townsend, Colonel, Corps of Engineers, U. S. Army.

No. 49. The tunnel towboat as used on the Trent, by F. Rayner, general manager of the Trent Navigation Co., Nottingham.

No. 51. Statement concerning motor boats on Russian rivers, by H. Merczyng, Professor at the Institute of Engineers of Lines of Communication, St. Petersburg.

At the International Congress of Navigation, held in Philadelphia, Penn., May 23-27, 1912, it was the practice to cover any one subject by papers from a number of representatives of different countries and to have these reports summarized by one man, known as the "General Reporter." We give below abstracts of some of the more important of these summaries, under the authorship of the respective "General Reporters."

No. 47. Side-wheel and Tunnel Boats — The commerce on the Rhine system in the year 1909 amounted to 58,390,068 tons. Of this traffic the Rhine proper carried 38,521,903 tons an average distance of 128.8 miles or a ton-mileage amounting to 4,962,000,000. The Rhine fleet in 1908 amounted to 1318 steamers and 9759 tow barges and other cargo vessels with a total capacity of 3,557,666 tons. Of the steamers 69% are tugs and 16% (including 47 Rhine sea-steamers) are cargo carriers.

Owing to the building facilities afforded by the Dutch ship-mortage banks, the Rhine has suffered from an over supply of barges and it has become the earnest endeavor of German shipbuilders to provide a light-draft towboat which shall be powerful and still relatively cheap. As the result of their studies in this direction the Schiffs-Maschinenbau A. G. of Mannheim, produced the twin-screw tunnel steamer "Gebr. Page X," this boat being a development from numerous smaller tunnel steamers built by the same company.

In order to finally determine the relative efficiency of the tunnel and side-wheel types of towboats, comparative runs were made upon the upper Rhine with the "Gebr. Page X" and the "Bavaria," a side-wheel boat of about the same power. The results of this test were plotted and finally reduced to the actual cost per lb.-mile-hour towrope pull as produced by each type of boat, the initial investment, fixed charges, wages of crew and coal consumption and all conditions of operation being considered.

The two boats are compared in detail, the tunnel boat having a mean i.hp. of 840 and the side-wheel boat 970. The length and minimum depth of the course over which the "Gebr. Page X" was run were 20 miles (32 km.) and 10 ft. 6 in. (3.20 m.), respectively, while the draft of the boat was 4 ft. 11 in. (1.50 m.), or 9¼ in. greater than normal. The corresponding figures for the trial of the "Bavaria" were: Length, 15 miles (24 km.), minimum depth 13 ft. (3.98 m.) and draft 3 ft. 11¼ in. (1.20 m.).

The towrope pulls, speeds and indicated horse-power were taken at intervals throughout the run and the results shown graphically. The "Page" towed 1450 tons of freight and the "Bavaria" 1800. The towboats themselves were tested for resistance at the speeds chosen for purposes of calculation, and for the "Bavaria" the results were added to the towrope pull. The total mean pull of the "Bavaria" at its mean speed was then reduced to the corresponding pull at the "Page's" mean speed and its own resistance at that speed subtracted.

There is thus left a towrope pull at 7.8 miles (12.5 km.) per hour of 15,241 lb. (6913 kg.) for the "Bavaria" at 970 i.hp. or 15.72 lb. (7.13 kg.) per i.hp., as against 10,800 lb. (700 kg.) for the "Gebr. Page X" at 840 i.hp. or 12.85 lb. (5.83 kg.) per i.hp. A comparison of these figures shows the "Bavaria" to apparently develop 22% more pull per i.hp. than the "Gebr. Page X," or stated conversely, the "Page" apparently developed 18% less than the "Bavaria."

On account of the finer lines of the "Bavaria," the author makes a correction by using for the denominator of the fraction the (i.hp. minus e.hp. of towboat alone), and the respective towrope pulls per e.hp. then become 18.58 lb. (8.43 kg.) and 15.76 lb. (7.15 kg.) and the percentages described above become 17.9% and 15.2%. A still further correction is made for the depths in which the tests were run and for the excessive draft of the "Page," using, as a basis, the speed of this boat at normal draft and with the same tow. A resulting towrope pull of 14.93 lb. (6.77 kg.) per i.hp. is found for the "Gebr. Pape X" against 14.88 lb. (6.75 kg.) per i.hp. for the "Bavaria" both at a speed of 8.04 miles (12.94 km.) per hour.

The report next shows the relative costs of producing 1 lb. (0.45 kg.) of towrope pull at 7.8 miles per hour with the two types of boats, considering initial cost, interest, amortization, wages, fuel and lubricants. A condensation of these figures is set forth in the following table:

		"Gebr. Page X," 840 I.h.p.	"Bavaria," 970 I.h.p.
1.	Initial cost	$44,960	$77,750
2.	Interest, amortization and insurance, 10.524 per cent.	4,732	8,182
3.	Fixed charges (above) per month	394	683
4.	Wages per month[1]	251	414
5.	(3) + (4) per month of 300 hours	645	1,097
6.	Fixed charges and wages per hour	2.15	3.66
7.	Cost of coal per hour at $5.00 per ton	4.08	4.71
8.	Lubricants and waste per hour	0.16	0.19
9.	Total cost per hour	6.39	8.56
10.	Cost per hour per lb. of towrope pull at 7.8 miles per hour	0.00059	0.00056
11.	Cost per ton of of freight	0.00056	0.00061

[1]The author charges the "Gebr. Page X." with only 2 firemen and 2 sailors, as against 6 firemen and sailors for the "Bavaria."

The author closes his paper with the conclusion, drawn from the foregoing figures, that the tunnel boat is at least the economical equal of the side-wheel steamer and cites the further advantages of low first cost, minimum width for use in narrow rivers and canals and reduced cost of maintenance.

No. 48. Traffic on the Mississippi — The water tonnage of New York Harbor for 1906 was 113,969,335, of which over 55,000,000 tons was harbor traffic originating in the system of rivers around New York, of which the Hudson, with a navigable length of only 150 miles is the largest. The corresponding tonnage of St. Louis, with 3700 miles of river through the great grain, coal and manufacturing districts of the middle of the continent, was, in the year 1910, 191,965 tons, having fallen from 2,120,825 tons in 1880. Most of this traffic having been absorbed by the railroads, the paper reports the result of an investigation into the causes of the change.

Authority is quoted to show that the tractive force necessary to draw a given tonnage on a railway decreases as the size of the units increases and that with cars weighing 75 tons, loaded, a locomotive can haul 200 tons of freight 5 miles per hour with an expenditure of energy not exceeding that required to propel the same tonnage in a canal boat at a rate of 2 miles per hour. The coal consumption per ton of freight per mile for various first-class roads is shown to vary from 0.067 to 0.073 lb. at about 18 miles per

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hour and to average 0.110 lb. at about 36 miles per hour. Over a mountainous stretch, an average consumption of 0.115 lb. was reached at a speed of only 5 miles per hour.

From these figures it is concluded that to haul freight on a river which is paralleled by a railway, the freight boat or tow must be prepared to compete with a train consuming 0.15 lb. per ton mile. The cost of barges and towboat will be in the neighborhood of $40 per ton of capacity for a 2000-ton tow, as against $30 for cars and engine. On the other hand the railway track costs about $58,000 per mile to build and $1,324 per year to maintain.

On some railroads coal is being moved at $0.0023 per ton-mile, which was the average freight rate on the Great Lakes in 1887.

The coal consumption per ton-mile of coal handled on the Monongahela River is quoted as being from 0.217 to 0.280 lb., including that necessary to return the empty barges. On the Ohio and Mississippi, the "Sprague" has towed 48,000 tons of coal from Louisville to New Orleans and returned the empty barges on an expenditure of 0.066 lb. of coal per ton-mile of coal carried down. The cost of the carrier is given in this case as $0.037 per ton-mile of freight towed per day.

Against this cost of the carrier, the cost of the river packet is given as $4.40 per ton-mile of freight carried per day, and the cost of a 2500-ton freight train is $0.30 per ton-mile per day for a mountainous district. The railroads' fixed charges are: 7% on the cost of the road bed and 18% on equipment. The fixed charges for the packet amount to 21%. Assuming for the railroad one 2500-ton freight train each way per day the fixed charges work out at a lower figure than those of the packet. It is, however, only in thickly settled portions of the country that such trainloads can be obtained, the average for 1909 being 362.57 tons, and as the load decreases the efficiency of the railroad becomes rapidly less.

Finally, the influence of lack of terminal facilities for river traffic is shown and river conditions compared with those on the Great Lakes.

The author's most important conclusions are:

1. That the problem of the "Utilization of the Navigation of Large but Shallow Rivers" differs in different countries and is a function of the extent and efficiency of their railroad systems.

2. To obtain supremacy for river navigation, it is necessary to utilize the towboat and barge method rather than the freight-carrying steamboat.

3. With this method (towboats) and a 9-ft. channel, as great economies of transportation may be obtained as on the Great Lakes; and with a 6-ft. channel greater economies are possible than at present obtain on railroads, if equal terminal facilities are provided.

4. A revolution in the means of propulsion by the introduction of electricity or liquid fuel will not revive river commerce where the railroad can utilize the invention as cheaply as the boat, although the motor boat for subsidiary lines will reduce the cost per ton-mile, owing to the small crew required.

5. The utilization of the navigation of large but shallow rivers is a factor of the density of traffic and of population.

No. 49. TUNNEL BOATS ON THE TRENT—The tunnel type of towboat, as used on the Trent, is described and the particulars of one of these boats, owned by the Trent Navigation Co., are given. The author reaches the conclusion that the hinged flap in a tunnel will yield an increase of speed of one knot beyond that obtained by a boat of the same dimensions and power when equipped with the usual type of tunnel.

No. 51—Conditions on Russian rivers are reviewed and the statement made that Russian designers have been forced to the saving of weight by the adoption of the internal-combustion engine.

A list of motor boats in use on these rivers is given, and a table of the most suitable dimensions for boats of 12 in. and 14 in. draft. The construction of boats in detachable sections is advocated in order to provide means of reducing draft by the addition of one or more sections.

COMMENT—The conclusions drawn by the authors of the foregoing papers appear to be clear and well sustained by the facts presented.

Reinforced Concrete in Hydraulic Works

BY JOHN STEPHEN SEWELL*

Five communications on this subject have been submitted as follows:

No. 31. M. Jacquinot, Chief Engineer of the Ponts et Chaussées, Chaumont (Haute-Marne), France.

No. 32. Mr. R. W. Vawdrey, A. M. I. C. E., Portscatho, Parkhill Road, Sidcup, England.

No. 33. The Hungarian State Water Survey.

No. 34. M. Mederico Perilli, Chief Engineer of the Genio Civile, Ravenna, Italy.

No. 30. Mr. Richard L. Humphrey, President, National Association of Cement Users, United States of America.

[All five of these papers were summarized by Major Sewell, and the summary is followed by a general résumé of the subject, which is printed below.—ED. ENG. NEWS.]

It is quite apparent that the use of concrete, both plain and reinforced, for all classes of hydraulic works, is making rapid headway, especially in the United States, France and Italy; and that opposition to its use is disappearing elsewhere.

Strictly speaking, the application of reinforced concrete to hydraulic structures is the only subject under discussion. All of the objections to the use of plain concrete in such cases are included within the greater number that have been urged against reinforced concrete. What follows, therefore, will apply especially to the latter material, and to the conditions surrounding its use in connection with works of inland navigation.

The objections that have been raised against the use of reinforced concrete for the purpose under discussion relate to the durability of the concrete itself, to its resistance to abrasion, chemical action, and freezing in contact with water, and to the durability of the reinforcement under various conditions.

DURABILITY OF CONCRETE—A few years ago, difficulty was sometimes experienced in securing a portland cement that was sound and secure against disintegration due to chemical changes within itself, after setting. This difficulty is now easily avoided, and if aggregates are selected of durable and inert materials, there can no longer be any doubt that concrete is in itself a thoroughly durable material, quite secure against disintegration due to action originating within the mass itself.

RESISTANCE TO ABRASION—Concrete, as a rule, exhibits more chipping and chafing under impact and abrasion than masonry of the harder stones. But this damage is usually only superficial, and rarely threatens the integrity or continued usefulness of the structure. In many cases, it is more resistant than any other form of masonry available within practicable cost limits; the best of masonry is liable to be disfigured under impact and chafing, and there is very little of it in existence subject to these conditions that does not show the wear and tear. In any case, the trouble can be avoided at moderate expense, by means of strips of steel or timbers applied in the proper manner. This objection to concrete is therefore not a valid one, for it can be removed by simple and practicable methods.

RESISTANCE TO CHEMICAL ACTION—Under this head may be included atmospheric agencies, the action of sea water, of sewage, and of acidulated water. So far as atmospheric agencies are concerned, there is too much concrete which has successfully withstood them, to leave any room for discussion. It is merely a question of good materials and workmanship, including proper mixtures to secure a dense and impervious mass.

A great many concrete structures exposed to sea water have suffered from extensive and rapid disintegration. It was at first supposed that this was due to the action of the sea water on some constituent of the cement, probably the

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free lime. Some experiments and investigations have seemed to indicate that the addition of trass or puzzolana to the cement would prevent this action, by satisfying the free lime. But it also appears probable that, as Mr. Humphrey states, a dense, strong and impervious mixture allowed to harden before exposure in place, is in itself sufficiently resistant, whether the preliminary disintegration has been due to freezing when saturated with water, or whether it is due entirely to chemical action. The requisite density and strength can best be obtained with a well balanced and rather wet mixture.

There is no doubt that wet mixtures will give denser concretes than dry ones, no matter how thoroughly the latter may be rammed. The use of dry mixtures had its origin, no doubt, in laboratory experiments, such as those of the late General Gilmore of the United States Corps of Engineers, which indicated that dry mixtures, when well rammed, produced cubes of greater strength than wet mixtures having the same proportion of cement and the same aggregates.

The use of wet mixtures originated, in this country, at least, from the fact that they require little ramming, make a smoother finish against forms, and generally cost less in place than dry mixtures well rammed. The practice of using wet mixtures was initiated by contractors, who were actuated by motives of economy, and made its way slowly against the opposition of engineers. It is fortunate, in this case, that the cheapest method is also the best; but it is only within a few years, at the most, that engineers have been fully convinced. Mr. Humphrey's warning against an amount of water sufficient to separate the ingredients should not, however, be overlooked.

It seems certain that there are many cases of concrete structures which are exposed to sea water in both warm and cold climates, and which are behaving satisfactorily. One successful example is of more value as evidence than a hundred failures; for it proves that success can be attained. If the necessary precautions are not yet thoroughly understood, it is only a question of further trial and study when they will be. The conclusion seems justified that exposure to sea water is not necessarily fatal to the use of concrete. As inland navigation may begin before the salt water ends, this point is germane to the subject under discussion.

Navigation canals may carry domestic sewage, factory waste, and acidulated water from mines, since all of these ingredients are to be found in the waters of streams from which canals are fed. So far as navigation works are concerned, the deleterious ingredients will generally be so diluted, that their action will be very slow, and probably the same precautions that suffice in the case of sea water will serve the purpose here also. Should cases arise where the deleterious ingredients exist in greater proportion, it is probable that any kind of masonry would suffer more or less, by action upon the cement in the joints, if in no other way. Limestone is probably more commonly used than any other natural stone for hydraulic works, and it would suffer in much the same way as cement. Many cases will arise, also, where there is no suitable stone within easy reach, and if concrete is not used, the work may be impracticable.

M. Jacquinot describes a reinforced-concrete conduit for carrying sewage; many large sewers of reinforced concrete have been constructed in the United States. The inverts are sometimes protected by other masonry—often of vitrified bricks or blocks—and sometimes by pitch or other coating. Many reinforced-concrete chimneys have also been built without a lining and they seem to resist the chemical action of the flue gases, and, in some cases, of the gases from smelters, very well. Melted paraffin could be applied to the finished surface and driven in by heat; while it is not known that this method has been tried, it is known to be inexpensive, and should greatly increase the resistance to acids. Finally, there are many methods of waterproofing masonry, any one of which ought to protect the concrete from contact with acidulated water or sewage and therefore from damage. It appears, therefore, that even the existence of these agencies need not be a fatal objection to the use of concrete, for their activities can be prevented at practicable cost; only in extreme cases would such protection be required in navigation works, and even then, the protected concrete may easily be the least expensive material available.

FREEZING IN CONTACT WITH WATER—Resistance to damage from this cause seems to be merely a question of density and strength. The same thing is true of stone and bricks. That concrete can be made sufficiently impervious and strong is demonstrated by many examples. Here again, a well balanced wet mixture, protected from washing out of the cement during the hardening process, is all that is required.

The damage from waters charged with alkaline salts seems to be of a mechanical nature, similar to that produced by freezing. While the problem here presented is not completely understood or solved, it seems probable that it is again merely a question of sufficient strength and density. While the problem is in process of solution, however, protective coatings are still within reach at moderate cost, and there is no valid objection to the use of concrete with proper precautions.

DURABILITY OF REINFORCEMENT—It is no longer open to serious doubt, that steel or iron thoroughly embedded in portland cement concrete will last indefinitely, as long as the covering remains intact. If it is exposed directly to the air, whether near the sea or not, it will inevitably corrode and ultimately destroy the structure. Careful design and good workmanship are all that are required to properly embed it in the first place. The only danger that threatens it thereafter is the danger of cracks, which will destroy the integrity of the concrete and open up a way for atmospheric moisture or water to gain direct access to the reinforcement. Such cracks might be due to shrinkage in setting, to expansion and contraction under changes of temperature, or to deformation under stress.

If the concrete is mixed wet and kept wet while setting, there is small danger of shrinkage cracks. In fact, cracks of this kind are rarely seen in practice. Light reinforcement is sometimes used with a view of preventing them, but it is not easy to see how it would be effective, for the shrinkage of the concrete would set up compressive stresses in the reinforcement, and it is not usually heavy enough to resist them effectively. However, the danger is more imaginary than real, and an abundant supply of water and protection from evaporation during setting will insure a successful result. If the concrete is reinforced for serious structural duty, the reinforcement will generally be heavy enough to prevent shrinkage cracks, even if they tend to form in exceptional cases.

Cracks due to expansion and contraction after setting are brought about probably by a slight slipping of the mass on its bed during expansion, and by the excess of frictional resistance over the tensile strength of the concrete during the subsequent contraction. This trouble can be overcome by proper reinforcement, but it would be well to divide a long wall or other structure into sections, so that each could act as a unit.

Cracks due to deformation under stress will occur only when the reinforcement is stressed so that the strain exceeds the limit of extensibility of the concrete. This can be avoided by proper design and workmanship. This danger makes it inadvisable to utilize the high working stresses, otherwise permissible in high-carbon steel, since the modulus of elasticity is no greater than with low carbon or medium steel. If working stresses are kept well within the limits allowable for mild steel, there is no danger of cracks in the concrete.

It appears, therefore, that it is entirely possible to maintain the integrity of the concrete coating and therefore to prevent corrosion of the reinforcement, and there is no valid objection on this score.

Some laboratory experiments have caused some engineers to fear that reinforced concrete subjected to hydraulic

pressure will have the bond between the concrete and the reinforcement destroyed, so that the two materials would cease to act in unison. But the old water tank cited by M. Jacquinot is in itself sufficient to settle this question favorably, even if there were not available the evidence of the numerous conduits sustaining high pressures cited by the same author, and the evidence of similar structures in this country.

Mr. Humphrey refers to the corrosion of reinforcement exposed near the sea. It is also a fact that if the concrete is mixed with sea water, or with sand from the sea beaches, or if it has salt mixed with it, and it is subsequently exposed to dampness, the reinforcement will corrode. This was shown by examples on a rather extended scale in some fortification work on an island in Long Island Sound, some years ago. The writer, then in the service of the United States, inspected this damaged work, and afterward had two experimental slabs of reinforced concrete made with a view of finally testing the matter. They were of identical composition except that one was mixed with sea water and one with fresh water. They were exposed to the weather on a roof in Washington, D. C., for some months. At the end of that time, the reinforcement in the sea-water slab was badly corroded, while that in the other one was entirely untouched. It is of importance, therefore, that the ingredients used in mixing concrete for hydraulic work should contain no corrosive material in themselves, if the concrete is to be reinforced.

CONCLUSIONS—The conclusion seems justified that all of the objections that have been urged against the use of reinforced concrete as a material suitable for use in connection with hydraulic works are either imaginary, or can be overcome by practicable methods, and must have arisen at a time when the subject was not so well understood as at present. That this conclusion is justified is abundantly proven by the increasing and successful use of the material in permanent structures everywhere, as indicated by the papers herein reviewed.

The great advantage of reinforced concrete lies in the fact that it is capable of withstanding stresses due to transverse strains, tension, and shearing. All the forms that could be executed in steel or timber can be closely imitated in reinforced concrete, which is immune from corrosion and decay. This makes it possible to adopt designs wherein the structure acts by its structural resistance and not by dead weight, and even the material to be retained and held back may be made, by this means, to add to the stability of the work as a whole. Dead weights on foundations are diminished, difficult excavation is often avoided or lessened, and total costs often greatly decreased, as compared with structures formed of masonry in mass; in many cases reinforced concrete affords the only practicable solution of a difficult problem, and in nearly all cases, it affords a variety of desirable solutions not practicable in any other material.

A study of the successful applications of reinforced concrete submitted to the Congress by the various reporters appears to justify the adoption of the following conclusions:

Reinforced concrete combines the structural qualities of steel and timber with the durability of good masonry. It is subject to no form of deterioration which cannot be avoided by reasonable precautions. It is free from many of the limitations surrounding the use of masonry in mass; because of the greater latitude it affords in the design and execution of structures, it often yields the best and most economical solution, and in some cases the only practicable solution, of the most difficult problems.

When properly designed and executed it is, therefore, among the most valuable, if not the most valuable material now available for use in connection with hydraulic works of all kinds.

The Improvement of Inland Navigation Channels, Including Bank Protection

BY HENRY C. NEWCOMER*

Ten reports have been submitted on this subject, as follows:

No. 37. Herr Regierungs- and Baurat Bergius, in Oderberg (Mark).

No. 38. Mr. E. J. Marote, Chief Engineer, Director of the Ponts et Chaussées, Brussels, and Mr. Jules Descans, Principal Engineer of the Ponts et Chaussées, Antwerp.

No. 39. Major William D. Connor, Corps of Engineers, United States Army, Member of the American Society of Civil Engineers.

No. 40. Mr. L. Dusuzeau, Chief Engineer of the Ponts et Chaussées, Professor at the National School of the Ponts et Chaussées, Compiegne (Oise).

No. 41. Mr. J. A. Saner, M. I. C. E., Chief Engineer of the Weaver Navigation, Northwich.

No. 42. Mr. Antonio Castiglione, President, and Mr. Mario Beretta, Secretary of the Committee for Inland Navigation, Milan.

No. 43. Mr. A. R. Van Loon, Engineer of the Waterstaat, Bois-le-Duc.

No. 44. Mr. E. A. Wodarski, Engineer of Lines of Communication, St. Petersburg.

No. 44bis. Mr. Emile de Hoerschelmann, Councillor of State, Member of the Council of Engineers at the Ministry of Lines of Communication, Tsarskoie-Selo (near St. Petersburg).

No. 45. Captain G. Malm, of the Royal Corps of Bridges and Roads, Chief Engineer of Works of Construction of the Royal Administration of Hydraulic Motive Powers of Sweden, Stockholm.

Two of the reports, Nos. 39 and 44, cover in a comprehensive way the general waterway situation, while the rest have been devoted mainly to certain features, to recent improvements in design, plans for new work, or measures proposed for the further development or improvement of navigation facilities. They will be reviewed briefly in their numerical order.

No. 37, BY R. BERGIUS—An illustrated account is given of some recent improvements in canal construction in Germany, including a flight of four locks and a safety gate on the Berlin-Stettin waterway, a new arrangement of inverted siphon culverts under the Datteln-Hamm Canal, and several forms of canal-bank protection.

The four locks have lifts of 30.75 ft. and are separated by short reaches in which boats can pass. Side basins are used for saving water, and the flight of locks is operated so that a prism of lift corresponding to only one-tenth of the total drop represents the amount of water withdrawn from the upper reach. In order to facilitate the passage of boats in the intermediate reaches, the consecutive locks are not in direct line with each other, but are displaced laterally to the right by the width of the lock chamber.

The safety gate is a horizontally framed barrier of the vertical lift type supported by an overhead bridge. When lowered it closes the waterway. It is furnished with counterweights at each end and can be operated by a single man. It is provided with valves through which the empty reach of the canal can be filled.

The novel feature of the inverted siphon culverts consists in placing the ends of the pipes at different levels so that they will come into operation successively as the flood rises.

Several forms of canal-bank protection are described and five illustrated by drawings. Stone pitching about 8 in. thick, either with or without a footing, is generally used along the water line, sometimes placed on a 4-in. bed of gravel. Occasionally, where the water level is quite variable, flat slopes of 1 on 4 to 1 on 5 are used without any protection.

No. 38, BY E. J. MAROTE AND J. DESCANS—The authors submit the following conclusions:

1. In the canals where the navigation traffic is not intense, where the draft of the boats is low and where the circulation of screw propelled boats is nil, the slopes are generally protected by plantation of osiers, of reeds or alders on the berms placed several centimeters below the water level. As the traffic develops, the necessity of a more efficacious protection makes itself felt and recourse is had to simple or double puddling.

2. When steam or motor traction is used, erosion, due to current or wash which the boats cause, soon occurs in mobile soils. This is generally remedied by the construction about the center of the water level of a skirting supported on piles and surmounted either by turfing or by protection in hard materials either in masonry work or not. When the local circumstances allow of the lowering of the water during more or less

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long periods, stonework in masonry or dry is preferred, founded on a berm generally one meter below water and sometimes supported by a framework of piles and skirting.

3. When the navigation of screw-propelled boats is great and when the lowering of the water level is not admissible the consolidation of the banks is obtained either by the "Villa" system or by the driving down of a row of piles and of a sheet-piling frontage surmounted by a covering of hard material above the water level. The nature of the soil encountered does not appear to exercise a preponderating influence on the covering adopted. On the other hand, there is no doubt that the width of the navigable waterway has a great influence on the erosion of the banks and constitutes an important factor in the choice of the system of protection.

4. At the Congress of Vienna the relation between the immersed cross-section of a boat and the wetted cross-section of a canal has been fixed at ¼, in the supposition that the speeds of the boats would not be greater than 6 km. per hour. The more this relation is increased the erosive action for the same speed is diminished in intensity.

5. On a navigable river the stonework is exposed to action of the current with the exception of eddy water provoked by the moving of the boats. The only means of protecting effectually the concave banks seems to consist in the covering of them with natural or artificial stones in masonry or dry, supported on a simple berm either on stone pitching or on a framework consisting of piles with skirtings of sheet piling.

6. In the interest of navigation and with a view to suppress stoppages to navigation or at least to reduce their duration, the method of bank protection should be designed in such a manner as to permit their maintenance without lowering the water level either under cover of sheeting or an easily removable and fixed cofferdam.

No. 39, BY W. D. CONNOR—The author gives a general account of the inland waterways of the United States and of the types of works used in their improvement, illustrated by 38 figures. The Great Lakes with their connecting waterways and the lake harbors are first described, and then the rivers, and finally the canals. Bank-protection work is described for rivers, but not for canals. As the report covers such a wide field, it is necessarily brief in its presentation of the different matters, and possibly in a few particulars it may be open to question, but it forms a very convenient summary of the waterways and methods of improvement in the United States.

No. 40, BY L. DUSUZEAU—This report is devoted to a brief discussion of the measures adopted or proposed in France for the suppression of general and periodical stoppages of traffic to make repairs, and for the methodical organization of waterway transportation. Reference is made to Mr. Colson's contention that railways are superior to waterways except under conditions that are specially favorable to the latter, but this conclusion is not considered fair, as it is said to be based on a comparison between railways having a very highly developed organization of traffic, and waterways on which the traffic is organized very poorly, if at all. The public welfare demands that water transportation be developed to the highest possible degree, and further improvement is to be sought mainly in the methods of operation.

The periodic closing of the waterways in order to make repairs has a disastrous effect upon traffic and is really unnecessary for their maintenance. It tends to postpone even those repairs that can be made without interference with traffic, and leads to emptying canal reaches or lowering the water without sufficient cause. Occasionally there will be some repairs that must be done in the dry, but these can be limited to short stoppages of infrequent occurrence. This has been demonstrated by six years' experience on canals of heavy traffic.

The waterways suffer from the present lack of system in canal traction and the absence of any organization of the traffic that would enable shipments to be made at regular rates on connecting lines and that would adjust the distribution of boats to meet the varying needs of different localities. The author believes unrestricted competition is best on large rivers, but on canals there should be a regulated monopoly of traction. There should also be established a commercial bureau or central body to serve as an intermediary between the various interests concerned in the traffic, which now lack coördination.

The measures proposed are as follows:

A. General stoppages must be abolished—at any rate on much frequented navigable waterways.

Such waterways should be made fit to undergo this new regime, for which purpose the following measures should be taken:

1. Put the under-water masonry into perfect order.

2. Choose a type of slope protection such as can be installed and maintained by means of movable dams of simple construction and at low cost.

3. Adopt a design or rigid iron lock gates without any woodwork about them, as wood continually requires repairs; prefer a means of closing sluices and gates which does not involve the use of delicate parts under water; such as the gate with a vertical heel post, and, especially, the balanced lifting gate.

4. For weirs in canalized rivers, give up those with needles which lie down on the floor and which floods and floating ice damage every year, and have recourse to great balanced sluices generally worked by steam or electricity, which can also, if the need arises, be worked by hand.

5. Make provision in the inverts and the lock heads for appliances to which cofferdams can be easily fitted so as to get at some part or the whole of these works without lowering the water in the reaches.

B. 1. The installation on navigable waterways, having an average traffic of over 20 barges a day, of haulage services monopolized by the State itself or under contract from it, with fixed charges—in connection with, and having similar rates to, the various systems in [illegible] State and in the neighboring ones.

2. The appointment of special [illegible] lishments which would form a link between the shippers, the consignees and the boatmen.

3. Unification, in neighboring States, of the general principles governing police conservancy regulations on navigable waterways and the laws applicable to demurrage charges on inland navigation.

No. 41, BY J. A. SANER—The author gives a brief statement of the recommendations of the Royal Commission on Canals and Waterways and submits a 40-page digest of the Commission's report which he prepared for the navigation company of which he is chief engineer. Nationalization of the waterways appears to be necessary for their development, though this involves state competition with the railways built by private capital.

No. 42, BY ANTONIO CASTIGLIONE AND MARIO BERETTA—This report is devoted to an explanation of the Bertolini law concerning inland waterways in Italy, after stating the circumstances leading up to its enactment. It is the outcome of an agitation for improved navigation that has been in progress for about ten years. The state has had control of navigable waterways, but they were allowed to deteriorate, and many proposals were made calling for the restoration of old channels and the provision of new ones. The principles governing the apportionment of necessary expenditures, as well as the determination of a program of procedure, were finally settled, after much discussion, by the passage of the Bertolini law in January, 1910.

This law separates the navigable waterways into four classes. The first class includes those in which military uses predominate; the second, those that lead to seaports and serve extensive areas; the third, those that connect important centers, but fall short of the other conditions mentioned above; the fourth, all others. The entire expense of works of the first class is to be borne by the state. The restoration and maintenance of works of the second class are also borne by the state, but new works of this class are to be paid for, three-fifths by the state, and two-fifths by the interested provinces and municipalities. The state contributes two-fifths to works of the third class, and between one- and two-fifths to works of the fourth class. This distribution of costs, placing part of the burden upon the interests directly benefited, is expected to have a notable effect in eliminating unworthy projects, and largely relieves the state from the necessity of determining the works to be undertaken by assigning the initiative to the districts that desire waterway improvements, and are willing to share in their cost.

The taxation of riparian lands appears

to be just in principle, but will require great care and judgment to avoid abuse. Lands may even be damaged in some cases, and reference is made to the numerous complaints on this score from agricultural interests along the Dortmund-Ems Canal. Considerable stress is laid upon the importance of the right to expropriate land for all purposes relating to the waterway, even for private industries that are dependent upon it. In no cases are tolls to be made a source of profit, their only purpose being to meet the necessary costs of improvement and maintenance.

Provision is also made for the formation of companies to advance funds for desired new works, and to be repaid in some time not exceeding 50 years, and for granting concessions to make improvement, thus affording an opportunity for private enterprise to engage in such work.

Already a promoting company has begun plans for a waterway from Milan to Venice for boats of 600 tons.

No. 43, BY A. R. VAN LOON—Bank protection in the Netherlands was quite fully discussed in a report by Mr. Wortman at the Hague Congress in 1894, and there has not been much change in methods since that time. The greater intensity of traffic, however, has led to strengthening the defenses in some particulars, and this report is devoted mainly to a statement of these details, illustrated by eight figures.

There has been an increase in the speeds allowed on the canals from Amsterdam to Ymuiden and from Ghent to Terneuzen. The change is greatest for boats of medium draft, and experience indicates that no further increase could be allowed.

No. 44, BY E. A. WODARSKI—This report, like Major Connor's general account of waterways in the United States, is a very condensed statement, and no attempt will be made to summarize it. It may be of interest, however, to note that while the author attaches great value to dredging as a means of improvement for Russian rivers, he also gives instances of successful regulation, and he considers this method essential on some streams and canalization on others, especially in cases where considerable increase in depth is sought.

No. 44BIS, BY E. DE HOERSCHELMANN—An outline is given of a preliminary project for a combined improvement of the Dnieper cataracts for navigation and power purposes. Former projects have had only navigation in view, though the possibility of power development was recognized. The rapids have a total fall of about 112 ft. in 41 miles, and remind one of the Muscle Shoals section of the Tennessee River, where there is a fall of about 131 ft. in 38 miles. It is possible for boats to navigate the Dnieper cataracts at times, going downstream, but not upstream. It is now proposed to canalize the cataracts by the construction of four dams. Each dam is to have a hydroelectric station and a lateral canal with double locks, 623 and 315 ft. long, respectively, and 52.5 ft. wide. The dams vary in height from 21 to 32 ft., and, using the hollow reinforced-concrete type, are estimated to cost but little more than half as much as dams of the ordinary solid type. The estimate for the navigation works under this plan amounts to about $14,800,000, which is only about one-fourth more than the last preceding estimate for improving the rapids, and these works will permit the development of more than 100,000 hp. at an additional cost of about $16,400,000.

No. 45, BY G. MALM—This report is devoted mainly to an account of the proposed reconstruction of the Trollhättan Canal which connects Lake Vänern with the sea at Gothenburg. Sweden has 3350 miles of navigable channels in lakes and natural waterways, and 897 miles in canals and canalized rivers.

The Trollhättan Canal was acquired by the government in 1904 and is the only one owned by the state. It was first completed in 1800, and was enlarged between 1838 and 1844 so as to accommodate boats of 346 tons. It has a length of about 52 miles and lies mainly in the Göta River, the total fall from the lake to the sea being about 145.5 ft. The traffic amounted to 1,233,589 tons in 1910. It is carried mainly in self-propelled vessels, some of which go to England and other foreign ports that are near at hand. The project to enlarge the canal was adopted after a thorough study by a commission to determine the most suitable dimensions for the prospective traffic, as related in the report by V. Hausen under the second question, first section. It was decided to give the canal at first a depth of about 14.4 ft. with provision in the permanent structures for subsequent deepening to 18 ft. The 15 old locks will be replaced by six new locks, four of which are in a flight at Trollhättan with a total lift of about 105 ft.

There is also given a brief description of the new canal begun in 1910 to connect Lakes Osten and Stora Lee, being practically a continuation of the Dalsland Canal which extends to Lake Vänern.

Improvement of Rivers for Navigation*

BY HENRY C. NEWCOMER†

The methods available are, regulation, dredging, reservoir construction, slack-watering by dams and locks and construction of a lateral canal.

*Considerably reduced from the original report.—Ed. Eng. News.

†Lieut.-Col., Corps of Engineers, U. S. A., Pittsburgh, Penn.

Ten papers have been submitted on this question, as follows:

No. 2, Herr Geheimer Oberbaurat Dr. Ing. Sympher, Wilhelmstrasse, 80, Berlin W. 66.

No. 3, E. Lauda, Diplomierter Ingenieur, Sektionschef im K. K. Ministerium für öffentliche Arbeiten in Wien.

No. 3, bis, Bohuslav Müller, Ingénieur en chef du Gouvernement Imp. et Royal de Boheme, attaché a la Commission pour la canalisation de la Vltava et de l'Elbe en Boheme, Prague.

No. 4, Major Wm. W. Harts, Corps of Engineers, U. S. Army, Customhouse, Nashville, Tenn.

No. 5, Wm. B. Landreth, Civil Engineer, formerly Special Deputy State Engineer, 20 Gillespie St., Schenectady, N. Y.

No. 6, Mr. Kauffmann, Chief Engineer of the Ponts et Chaussées, rue Dugommier, 9, Nantes.

No. 8, Eugene de Kvassay, Ministerial Counsellor, Chief of the Hungarian State Water Survey.

No. 9, Charles Valentini, Chief Engineer of the Genio Civile, at Bologna.

No. 10, R. H. Gockinga, Chief Engineer of Waterstaat, at the Hague and Messrs. H. Baucke, E. Van Konynenburg and Jonkheer C. W. Van Panhuys, Engineers of the Waterstaat at Nimegen, Maestricht and Zutphen.

No. 11, V. E. de Timonoff, Professor at the Imperial Institute of Lines of Communication, Director of Statistics and Cartography of the Lines of Communication, etc., 7 Perspective Ismailovsky, St. Petersburg, and G. H. Kleiber, Engineer of Lines of Communication in charge of dredging, Stoliarny, 11, St. Petersburg.

No. 2, BY LEO SYMPHER—This report is devoted mainly to a discussion of the new plans for the improvement of the Weser and a part of the Oder by regulation, assisted by reservoirs.

The Rhine, Weser and Elbe have been improved entirely by regulation. In the period from 1875 to 1905, the density of traffic increased from 720,000 to 5,800,000 metric tons per kilometer on the Elbe, from 80,000 to 480,000 on the Weser, and from 1,560,000 to 11,400,000 on the Rhine. In the upper reaches of the larger rivers and in the smaller rivers, where regulation could not provide a channel that would satisfy the traffic requirements, improvement has been effected by canalization, as in the upper Oder, the Fulda and the Main. While canalization is the surest, and sometimes the only means of providing an adequate channel—or at times a lateral canal—the delays in passing through locks are objectionable, and regulation is preferable where it can accomplish the desired result. Reference is made to the great cost of canalization, amounting to about $13,000 per mile on the Main, $147,000 on the Oder, $52,000 on the Fulda, and $186,000 (estimated) on the Lippe. The possible damage to private property caused by raising the water level must also be considered.

In some cases, especially where canalization has given a greater depth on the upper reaches of a river than regulation alone can provide on the lower, an alternative method of improvement has been suggested, which consists in supplementing regulation by storage reservoirs to increase the low-water flow. Works of this kind have been planned in Germany on the Oder and the Weser.

Since 1888 the upper Oder has been canalized from the harbor of Cosel to Breslau so as to give a minimum channel depth of 59 in. for 400-ton boats. Below Breslau the channel has been regulated, but at some specially unfavorable points the extreme low-water depth is scarcely 31.5 in. It was proposed to canalize the Middle Oder also, but this plan was abandoned in favor of additional regulation assisted by storage reservoirs. Experimental improvement of three trial stretches of the river has led to the conclusion that a depth of 53 in. can be secured by further regulation and a reservoir capacity of about 5,300,000,000 cu.ft. This depth can be maintained except during seasons of extremely low water, such as 1904, and will permit loading the 400-ton boats to ¾ of their capacity. The additional regulation required and the two reservoirs are estimated to cost about $60,000 a mile, as compared with $147,000 a mile for canalization of the upper Oder.

The author devotes his principal attention to the Weser, on which river this new plan of supplementing regulation by adding to low-water flow from storage reservoirs was first adopted in Prussia. The Fulda, which unites with the Werra to form the Weser, has been canalized and accommodates 400-ton barges from Münden to Cassel, but the Weser has been improved mainly by regulation. The mean low-water discharge, varying from about 780 cu.ft. per second at Münden to 3350 cu.ft. per second at Bremen, has permitted the formation of a channel with a depth varying from about 26 in. to 40 in. between these two points. On these low-water depths the 400-ton barges which draw about 12 in. light and 59 in. fully loaded can be loaded from ⅓ to ½ of their full capacity.

These conditions were considered fairly satisfactory, in view of the comparatively small commerce on the Weser, until the Rhine-Elbe Canal was projected, when the complete canalization of the Weser was also studied. This would require 51 locks and dams, 26 in the 84 miles above Hameln and 25 in the 144 miles below that place to Bremen. It was concluded that the canalization was not justified above Hameln, but was necessary below it in order to permit the Rhine-Elbe Canal to be fed from the Weser, and also to provide Bremen with a good navigable connection with the canal. Bremen was to pay for the work below Minden where the canal crosses the Weser, and the Prussian government for that above Minden to Hameln. When it was decided to terminate the canal at Hanover, Bremen was no longer willing to contribute so largely to the work, and it was decided to give up the plan for canalization of the river, and to secure the necessary feed water for the canal, together with some improvement of the navigable channel and other incidental benefits in reduction of floods and production of water power by the construction of two reservoirs on the Eder and Diemel, having capacities of about 7,140,000,000 and 700,000,000 cu.ft., respectively, or a total of 7,840,000,000 cu.ft. at a cost of about $700 per million cu.ft.

The author submits the following conclusion:

> In the preparation of plans for the rendering navigable of rivers, it is advisable, under conditions that appear suitable, that the question be gone into, whether, and in what degree, a regulation made by the aid of make-up water from reservoirs is possible, and whether it is to be preferred to a canalization.

No. 3, BY E. LAUDA—This report is devoted to a discussion of changes in condition of flood flow that will follow the improvement of the Middle Elbe in Bohemia as a result of reduction in volume of overflow or partial removal of the reservoir effect due to inundation of its valley. The Middle Elbe, extending from Melink to Jaromer, a distance of about 140 miles, is to have 29 locks and movable weirs, and the channel will be shortened about 50% by the removal of sharp bends and given a greater sectional area so that it is expected to have sufficient capacity for the destructive summer floods, while the fertilizing value of the greater spring floods will still be preserved. It is estimated that for the great flood of 1891, the inundation volume in the Middle Elbe would be reduced about 14% after regulation, and that this would be attended by a drop of about 16 in. in the flood height on the upper portion of the stretch under consideration, while the flood height would remain about the same on the lower portion, and be increased somewhat below Melink, the rise in flood level at Tetschen being estimated at about 12 in.

No. 3BIS, BY BOHUSLAV MUELLER—This report considers the question as to whether improvement of navigation by reservoirs could be substituted for canalization on the Elbe between Leitmeritz and Aussig. It is known that canalization would provide the desired depth for 770-ton barges. Regulation alone could not produce this result, because of insufficient low-water flow, and it was suggested that this deficiency could be made up by storage reservoirs. Prof. Harlacher had indeed proposed in 1893 extensive reservoir construction as a means of providing for navigation in both the Moldau and the Elbe in Bohemia, but the estimated cost of about $200,000,000 made the plan financially impracticable. While the full depth of channel desired could not be secured with the aid of reservoirs, it was thought that provision might possibly be made for sufficient water to carry half loads. An examination of a critical year, such as 1904, shows that the storage capacity required for this purpose, without further regulation, amounts to about 45,900,000,000 cu.ft. By additional regulation, however, the capacity required may be reduced from 40 to 80%. The larger amount could not be stored in some dry years, and the smaller amounts, together with the additional regulation, would cost at least twelve times as much as the canalization for the reach under consideration.

No. 4, BY WM. W. HARTS—This report discusses all of the various methods of river improvement, referring to the advantages and defects of each type, the conditions that favor its application and its limitations in promoting navigability, and illustrating the subject by references to work done on a number of streams. The author considers reservoirs but little applicable to the improvement of rivers for navigation. Aside from a few very exceptional situations such as at the headquarters of the Volga and the Mississippi, where a large storage capacity can be secured at a comparatively small cost, the expense of any adequate reservoir control of stream flow appears to be too great to be undertaken. The author submits the following conclusions:

> 1. Regulation in some suitable combination with channel excavation should always be first studied as a method of river improvement and adopted in all cases where economically applicable. It will be oftenest used wherever the funds available are small in amount, the increase of depth needed not great, the river flow comparatively large, the banks low and the width of the river considerable, velocities low and regimen more or less fixed.
>
> 2. Canalization with movable or fixed dams will be adopted wherever regulation with channel excavation is insufficient or unsuitable. It will usually be applied where the slope is steep, discharge small, and depths obtainable by regulation insufficient.
>
> 3. Lateral canals should never be selected for use unless imperatively demanded by the local conditions.
>
> 4. Reservoirs are too uncertain, too unsafe and too expensive for exclusive use in river improvement. They will seldom be relied on, except in special cases in connection with other enterprises where their use for industrial purposes warrants the cost and the water flow can be sufficiently controlled to operate beneficially on the channels.

No. 5, BY WM. B. LANDRETH—This report discusses briefly the methods of improvement by regulation and reservoirs, and then gives in considerable detail an account of the canalization of the Mohawk River as a part of the New York State Barge Canal. The greatest depth that regulation and reservoirs could provide would be 3 ft., whereas a depth of 12 ft. is desired, and canalization is therefore necessary. Even with canalization some storage is necessary in order to avoid interference with existing water powers, and two reservoirs are provided to give the necessary water supply. The author submits the following conclusions:

> 1. That the improvement of rivers by regulation is feasible for streams of stable regimen, if the proposed depth

is not greatly in excess of the unregulated depth on the bars or shallows; provided water for the increased flow at the regulated stages can be secured by storage or otherwise.

2. That the low-water flow of a stream and the practicability of adequately increasing it by storage are important, and often governing factors in determining the method of improvement to be used.

No. 8, BY EUGENE DE KVASSAY—This report presents the results of investigations on Hungarian rivers, showing that canalization is required to provide a navigable depth at low water, except on the Danube and a few larger tributaries, aggregating a total length of 1619 miles, where the depth of 2 meters is available at lowest water, or can be secured by regulation.

The Bega has a minimum discharge of 68 cu.ft. per second, which is increased to 127 cu.ft. per second by water brought from the Temes. A flow of 1236 cu.ft. per second is required to provide the depth of 6.5 ft. required for 650-ton boats. It was estimated that the cost of 21 reservoirs, holding about 3,390,000,000 cu.ft., would cost over $5,000,000, while the canalization of the Bega would cost about $1,200,000, and works of flood protection about $1,920,000. It was decided to canalize the river because this is cheaper and more certain in its results, and permits the water to be stored and used for other purposes, such as irrigation. Moreover, even if completely stored, it would not suffice to give the desired depth.

The question of water storage and reservoirs is of immense importance for Hungary; but it can only be solved in connection with irrigation and hydroelectric stations for generating power, as is proved by minute investigation.

No. 9, BY C. VALENTINI—This report discusses the various methods of improvement and the limits of their application. A radius of 1650 ft. is advised for curves to accommodate boats of 400 to 600 tons, diminishing to 660 ft. for a very low speed.

In order to avoid expense and lessen the risk of damage to riparian interests, it is well to adhere as closely as possible to the natural course of the river. The works of contraction should be longitudinal dikes in the hollows of bends and where the banks are much exposed, and spurs, dikes, or groins elsewhere.

Canalization with movable weirs is mentioned as another method of improvement, but the principal emphasis is laid upon its possible disadvantages, and especially its great cost and expense of operation and maintenance. Thus while canalization in Europe has ranged from $45,000 to $120,000 per mile, regulation has cost only from $34,000 to $64,000 per mile. Regulation, therefore, is preferable where practicable. The author states that the Po, the Tiber, and the Arno are now being improved primarily by dredging, to be supplemented later, if need be, by contraction works. As the slope increases, however, and the discharge diminishes, regulation becomes more difficult, and finally impossible, and must give way to canalization or a lateral canal.

The author's main conclusions may be presented in the following quotations:

The data relating to the gradient and volume of flow are the factors that contribute to establishing the limits beyond which the regulation of a river is no longer possible, and where, on the contrary, canalization becomes essential.

The gradient should not only be under the limit beyond which the force required for towing vessels upstream becomes greater than the effort of traction in the case of an equivalent railway train—since otherwise, navigation could no longer compete with the railways—but in addition, it should not exceed that beyond which erosion of the banks and bed of the river might occur.

On the other hand the volume of flow should always be sufficient, even when the water is low, to enable the cross-section to afford easy passage to two vessels of the highest tonnage admitted to navigate the river in question.

No. 10, BY R. H. GOCKINGA, H. BAUCKE, E. VAN KONYNENBURG AND J. C. W. VAN PANHUYS—This combined report gives many details concerning the physical conditions and the methods of improvement of the Dutch rivers, Waal, Yssel, and Meuse, illustrating certain limitations attending the different kinds of work. The usual method of regulation has been to form the low-water bed by longitudinal dikes and transverse groins or spurs. Dredging is employed to remove materials that resist erosion and to provide sufficient depth where the works of contraction are ineffective. High waters are confined by flood dikes and the land between these dikes and the river is usually protected against summer floods by smaller dikes.

The Waal and Yssel are in the delta of the Rhine, the Waal carrying about two-thirds and the Yssel one-ninth of the flow of the upper Rhine.

The Waal has a very important commerce, amounting to more than 20 million tons a year. The desired depth of channel is 3 meters (9.84 ft.), which is sufficient to accommodate barges of large capacity up to 3500 tons. The works of improvement have consisted of longitudinal and spur dikes, largely supplemented by dredging. The original widths adopted for the channel were found to be too great and have been reduced from time to time in order to secure more satisfactory and permanent results.

The four authors are not agreed on all points brought out in the report, but they are unanimous in presenting the following conclusions:

1. As a rule the standardization of a river for the benefit of navigation by a systematic and regular reformation of the cross-sectios is far more favorable than canalization or the cutting of a lateral canal. In fact, on a standardized river, the navigation will be free of all impediments, whereas on a canalized river or a lateral canal it will be interrupted with locks.

2. The standardization definitely defined under 1 is not always possible. This eventuality will occur when in regard to the slope and the required depth below ordinary low-water levels the flow of the river during dry summer months requires a narrowing such that the width becomes insufficient for the navigation in view.

3. Dredging should only be carried on in the upper reaches in conjunction with the regulation works. It may, however, be used as method of local independent improvement for the standardization, for instance when the ground along a navigable channel to be obtained consists of gravel, hard soils containing iron and clays.

No. 11, BY V. E. DE TIMONOFF AND G. H. KLEIBER—The method of improvement by dredging, proposed by V. E. de Timonoff in 1897, and advocated by him before the Congress at Paris in 1900, was first applied by Mr. Kleiber on a trial section of the Volga in 1901, and gave very satisfactory results. It has later been extended to other rivers and has easily accomplished the desired improvements, so that it is now regarded as the chief means of promoting the navigability of Russian rivers.

The dredges begin working soon after the flood subsides so that the bars are all in navigable condition when the period of low water arrives, and the dredges can then be employed on other work. The full benefits expected have not yet been developed on the upper and lower sections mentioned above, but on the middle section, the desired depth has been maintained for nine years. The increase in low-water depth due to dredging may be given as 2.6 ft. to 3.3 ft. There are a few permanent works of regulation, but these have been mainly a hindrance and are to be abandoned.

The gentle slope, large mean discharge, and the character of the floods on the Volga all favor the method of improvement by dredging. As a rule, the cuts do not have to be renewed every year. Less than half the bars in the second section require dredging each year, and the slow decline from flood stage permits the worst shoals to be treated in succession. The average annual cost of the dredging is said to be about $357 per mile of the river. Among the conclusions submitted by the authors are the following:

The method for the improvement of the navigability of rivers, by means of regulation obtained from permanent works, could not be applied successfully, with the needs of the navigation of today, without a very great use of dredging, both at the time of the formation of the regulated bed by the method adopted and during the later maintenance of the depths obtained. The method of canalizing rivers implies regulating the bed by means of permanent works and the use of dredging, as does

also the method of the increase of the low-water discharge and of the depth with the assistance of storage reservoirs; hence every improvement of the navigability of a river should begin, on principle, with procuring a dredging plant well adapted to local conditions and with dredging which, furthermore, is often able to give by itself alone a rapid and economical solution to the problem of the improvement of the navigability of the river in question so long as this problem be not complicated by exigencies of another sort.

In the case of rivers having a great discharge and a light slope, the method by dredging is the only really practical means for improving navigability and the only one able to satisfy all the demands of navigation without bringing on excessive expenditures of time and money.

SUMMARY BY COLONEL NEWCOMER—It is quite apparent that no single method of improving the navigability of rivers is generally accepted as having merit superior to the others. While conditions in one country give special prominence to one method, the situation in another country leads to different results. In some cases increased navigation facilities can be supplied in several ways, while in others any adequate improvement is practically limited to one form of procedure.

It is believed that more emphasis should be placed upon the financial side of the problem. Usually the improvement that it is physically possible to make varies through a wide range, depending not only on the method employed, but also on the amount that can be expended, and it is necessary to determine the cost that is best proportioned to the commercial benefits. As the waterway problem is essentially one of transportation, it may even be advisable in some cases to inquire whether the commercial needs may not be best satisfied by railroad construction. This consideration would, of course, apply with greatest force in those instances where the authorities or parties conducting the investigation are in a position to provide whichever form of transportation is deemed most desirable.

It may be of some interest to refer to the results of investigations made a few years ago concerning the improvement of the Ohio and Mississippi Rivers by dredging. The Ohio River was examined with a view to its improvement by canalization to secure depths of 6 or 9 ft., and also to determine whether these depths could be maintained by dredging in the lower part of the river, for a distance of about 190 miles, from Green River to Cairo, where there is a slope of about 3.7 in. per mile. The conclusion was reached that either 6 or 9 ft. could be maintained by dredging, and that the dredging would cost about 38% less than canalization for a depth of 6 ft., and about 60% more than canalization for a depth of 9 ft. The improvement of the Ohio by storage reservoirs has also received some consideration, and it has been ascertained that the entire runoff at Pittsburgh, if completely controlled, could not give the desired depth of 9 ft., as the mean annual discharge corresponds to a less depth than this.

It is well known that a depth of 9 ft. has been maintained by dredging in the Mississippi River below Cairo for a number of years, with occasional slight exceptions. Above Cairo to St. Louis, the project provides for a depth of 8 ft. to be obtained by dredging and regulation. The agitation for a deep waterway from Lake Michigan to the Gulf of Mexico led to an investigation of the practicability of securing a depth of 14 ft. in the Mississippi River below St. Louis. All methods of improvement were considered and Mr. Landreth has given the adverse conclusions reached with reference to the use of reservoirs. The Board of Engineers that conducted the investigation decided that dredging could maintain a depth of 14 ft. below Cairo and St. Louis. The estimated costs, however, ran very high, being about $12,300 per mile per year above Cairo and $3100 per mile per year below Cairo.

The divergence of views expressed, especially on the subject of dredging, makes it doubtful whether any conclusions can be framed that will meet with general approval in all respects, but the following propositions are submitted for the action of the Congress.

1. Under the widely varying requirements of navigation, and the very different physical conditions of slope, discharge, and nature of bed, no single method of improving the navigability of a river has superior advantages in all cases, but each may in turn be found most satisfactory under special conditions.

2. The choice of a method of improvement depends not only on the capacity of the stream for improvement by the different methods, but also on the volume of commerce to be benefited and the resulting cost of transportation, including interest on the cost of improvement, maintenance charges and the cost of carriage.

3. The slope, discharge and nature of bed and banks are the main factors determining the limits and the cost of improvement by the ordinary methods of regulation, dredging and canalization, and the prospective tonnage is the main factor determining the justifiable expenditure.

4. Regulation and dredging, either singly or in combination, are apt to be more uncertain and limited in their results than canalization, but they are usually preferable when the needs of navigation can be satisfied by these means. Otherwise it is generally advisable to employ canalization, using fixed or movable dams, depending upon the limitations imposed by flood conditions, and the requirements of navigation.

5. Lateral canals are usually less desirable than canalization, but may be required under some conditions.

6. Reservoir control of stream flow, sufficient to meet the needs of navigation, is usually impracticable within reasonable limits of cost, but in rare cases it may be used to advantage to supplement other methods of improvement.

7. It is desirable that the following steps be taken with a view to perfecting the methods employed for improving the navigability of rivers.

(a) That scientifically organized special studies be undertaken by sundry nations, on rivers with different regimens in order to observe the degree of navigability which it is possible to attain by the application of various methods of improvement and to determine the factors which govern the cost of the corresponding works.

(b) That hydrotechnic laboratories intended for the study, on small scale models, of the life of rivers become of more and more extended use, and that they be supplied with the means necessary to experiment with the various processes for improving the navigability of rivers and, in so far as possible, in connection with the studies and works carried out on the rivers themselves.

(c) That the resolution of the VIth Congress of Inland Navigation, voted at The Hague in 1904, be carried into effect, this resolution calling for taking up, in connection with rivers having but one current, the study of a short, clear formulary, which shall be sufficiently complete and include the information necessary to define the characteristic of every river studied, from the double point of view of its regimen and its navigation.

(d) That the question of the improvement of the navigability of rivers having but one current, completed by those of the laboratory experiments and of the formulary, be kept on the order of business of the next Congress of Navigation.

The Steam-Electric Power development, at a colliery of the Lehigh Coal & Navigation Co., for the long-distance transmission of electricity, has been carried forward to such an extent that contracts have been let to the General Electric Co., of Schenectady, N. Y., for three 10,000-kw. 11,000-volt three-phase 25-cycle horizontal turbine generating units with two 300-kw. turbine exciter sets and one 300-kw. motor-generator exciter. In Engineering News, Mar. 23, 1911, page 367, we noted the main points of this project. The power station is at Hauto, where mine waste will be burned. The development has been under the direction of the late W. A. Lathrop, president of the Lehigh Navigation Electric Co., of Philadelphia, and L. B. Stillwell, consulting engineer, of New York. It is now reported that slate and cement industries in Lehigh and Northampton Counties, Pennsylvania, will be supplied first. This territory will be from 20 to 40 miles distant from the Hauto plant. A complete power census of the industries in a radius of 50 to 75 miles from the main station has been taken and it has been found that 100,000 hp. is now being used.

Under the laws of Pennsylvania a steam-electric power company can operate only in a single township; accordingly charters have had to be obtained in Lehigh, Schuylkill, Carbon, Northampton, Bucks and Montgomery Counties for about 60 electric companies. All the subsidiary companies, however, will be under the consolidated management of the Lehigh Navigation Electric Co. It is expected to have the first installation at the central station in operation before 1913.

Livingstone Channel; Detroit River

By E. T. Lednum*

The comparatively narrow deep-water channel in the Detroit river, connecting Lake Erie with the upper lakes, causes a congestion of traffic of the larger steamers and has some dangerous points at which a number of accidents have occurred. To give greater depth and width for the modern large vessels, a new channel is being made in the river and is approaching completion. This is nearly 12 miles long and 300 to 450 ft. wide, with 22 to 24 ft. of water. Part of the work is done under water by drilling, blasting and dredging the rock, but a specially interesting feature is that at one point an area of the river bed has been inclosed by cofferdams, the area being then pumped out and the excavation done in the dry. This paper gives a general description of both the submarine work and the dry work.

*E. I. DuPont de Nemours Powder Co., Old Colony Building, Chicago.

On Oct. 7, 1907, bids were opened by the War Department (in the office of Col. Chas. E. L. B. Davis, Corps of Engineers, U. S. A., at Detroit, Mich.) for the improvement of the lower Detroit River known as the Livingstone Channel. This is about 11½ miles long, and is intended for the use of down-bound boats. It will relieve the congestion in the traffic and the attendant danger in the necessity of both up-bound and down-bound vessels using the channel between Bois Blanc Island and Amherstburg (Ont.), and over the Limekiln crossing. The tonnage of ores, grain, salt and lumber passing down, and coal and other supplies passing up during the season of navigation for 1911 aggregated about 65,000,000 tons, valued at several hundred millions of dollars. Up-bound vessels will continue to use the present channel, which will be deepened and improved. It is probable also that a dam will be built north from the upper end of Bois Blanc Island, between the two channels, because of the strong cross current.

The work consisted in excavating a channel through limestone bedrock, sand, clay, gravel and boulders or loose stones. For a width of 300 ft. all material is excavated and removed to a clear depth of 22 ft. by the subaqueous method and 23 ft. by the dry method (part of Section 2). This depth is below a naturally sloping water surface, the elevation of which, at the old Stony Island pier, is 6.27 ft. below a U. S. bench mark whose elevation is 577.524 ft. above mean tide at New York (levels of 1903). The work was divided into four contract sections.† [The situation is shown by the map, Fig. 1, which represents the conditions in April, 1912, and is from an official drawing furnished us by the U. S. Engineer Office. The new channel extends from the present deep channel at *C* to the 22-ft. contour line in Lake Erie, at *H*, crossing the older channel at the Detroit River lighthouse. The present channel is 600 ft. wide and 19 ft. deep from *J* to *L;* it is being improved to a depth of 22 ft. as far as *K*, and will be deepened to 21 ft. from *K* to *L* after the completion of the Livingstone channel. From *L*, the depth is 21 ft. to the 21-ft. contour of Lake Erie, with a width of 600 to 800 ft. as far as the intersection with the new channel, and then 400 ft. to deep water.—EDITOR.]

SECTION No. 1—This comprises a channel 300 ft. wide and 22 ft. deep from the present channel, at *C*, to Stony Island; the width being increased at the junction, as shown. On Oct. 31, 1907, the contract was let to the Great Lakes Dredge & Dock Co., at $1.98 per cu.yd. for excavation (bank measurement). The amount for the work was $609,820. Work was started Dec. 7, 1908, and continued until completion. The upper end, *C* to *D*, was excavated by the subaqueous method, but the south 1500 ft. was sublet to Grant Smith & Co. and Locher (contractors for section No. 2), and was inclosed by a coffer-dam and excavated in the dry. Material used in construction of the coffer-dam was dredged from the channel to the extent of about 215,000 cu.yd. The dam was completed Aug. 10, 1909. The channel is completed above the dam to the river, at *C*, Fig. 1.

SECTION No. 2 — This is excavated partly in the dry, in an inclosure surrounded by coffer-dams, with a channel 450 ft. wide and 23 ft. deep; the remainder is a dredged channel 300 ft. wide and 22 ft. deep. On Nov. 1, 1907, contract was let to Grant Smith & Co. and Locher. Excavation inside coffer-dam, $1.24 per cu.yd. for rock, 60c. for earth; excavation outside coffer-dam, $3.40 for rock, 60c. for earth; building coffer-dam, $25,000; removing old abandoned pier, $5000. The total for the work was $1,506,805. Work was started Apr. 14, 1908. The dredged portion, *E* to *F*, is completed. The entire dry section (including sections Nos. 2 and 3) will be completed in July, 1912, after the removal of the north and south coffer-dams which cross the end of the 300-ft. dredged sections. Fig. 2 is a cross-section on the line *A-B* (Fig. 1).

SECTION No. 3—This is a dredged channel 300 ft. wide and 22 ft. deep, from *F* to *G*, Fig. 1. On Dec. 23, 1907, contract was let to O. E. Dunbar and P. B. McNaughton. The price for excavation was $2.80 per cu.yd. for rock and 50c. for earth, both bank measurement. The amount for the work was $2,700,750. Work was started Apr. 9, 1908, and is to be completed by July, 1912.

SECTION No. 4—This is a dredged channel, from *G* to the 22-ft contour line of Lake Erie at *H*. Contract was let to G. H. Breyman & Bros., at 25½c. per cu.yd. for earth excavation, bank measurement. They sublet portions of the work to the Starke Dredge & Dock Co. and the Great Lakes Dredge & Dock Co. The material in this section was earth and was removed by dredging. The length of this section is about 29,000 ft. and widens from 300 ft. to 800 ft. It is dredged to a clear depth of 22 ft. and maintains these measurements to deep water in Lake Erie.

For the rapid and successful execution of the work under these contracts much credit is due to Col. C. McD. Townsend, Corps of Engineers, U. S. A. (who relieved Colonel Davis); to C. Y. Dixon, assistant engineer, and his field men; and to the contracting firms doing the work.

The following detailed description of the work relates to sections Nos. 2 and 3.

SECTION No. 2

On this work for a distance of 4300 ft. the excavation was made in the dry from an area surrounded by coffer-dams. This was known as the "dry section" and is shown on the map by the hatched lines (representing the coffer-dams). A distance of 3500 ft. on the south end was sublet to the Dunbar & Sullivan Dredging Co., and excavation was made by the subaqueous method. Fig. 2 is a cross-section of the part excavated in the dry.

The coffer-dams were begun on Apr. 4, 1908. They are in 18 ft. of water at the north end and 9 ft. at the south end. Their average height above the river level is about 4 ft. The dams were built by dredges where the material on the river bottom would permit. Where this was not feasible, earth excavated from the channel was loaded onto scows and dumped along the line of the dam until the material was of such height as to prevent further dumping. Deck scows were then used, with a derrick boat to

†The earlier part of this work was described in our issue of Dec. 17, 1908, and it is of interest particularly on account of having a large area inclosed by coffer-dams, pumped out and then excavated in the dry. This portion (including parts of contract sections Nos. 1 and 2, is shown by the hatched lines adjacent to Stony Island in Fig. 1.—Editor.

FIG. 1. GENERAL MAP OF THE NEW LIVINGSTON CHANNEL IN THE DETROIT RIVER

(The hatched lines adjacent to Stone Island represent the portion inclosed by coffer-dams, unwatered and excavated in the dry. The dotted lines show the present channel.)

Stony Island
Eva Neck
Cofferdam
West Spoil Bank about 30 Ft. high, 150 Ft. wide
Revetment
Part of North and South Cofferdams to be removed
Original 300 Ft. Channel
50 Ft. Widening
East Spoil Bank from 15 Ft. to 45 Ft. high, 200 Ft. to 280 Ft. wide
East Cofferdam
EL 571.25

FIG. 2. CROSS-SECTION OF 450-FT. CHANNEL EXCAVATED IN THE DRY

(Section on line A-B of plan, Fig. 1.)

unload them. The derrick boat was raised on spuds and had a long boom which extended across the scow. A cable passed over sheaves at each end of the boom and over a drum. Each end was fastened to a scraper, and as the drum was revolved by an engine, the scraper was operated across the deck of the scow, quickly unloading the material, as shown in Fig. 3.

The core of the north dam was rock and the remainder of loose material and mud, unloaded in the above manner. As this work progressed, it was seen that the entire area could not be closed before winter. A transverse coffer-dam was accordingly built about 1200 ft. upstream from the lower end of the dry section (Fig. 1), so that excavation could be carried on during the winter. The area thus inclosed (about 112 acres) had an average depth of water of 12 ft. The plant used for unwatering this area consisted of two 12-in. centrifugal pumps, and 49 siphons.

These siphons were 8-in. cast-iron pipes placed in an inclined position, with their lower ends in a sump and the upper ends emptying into the Detroit River. They were operated by compressed air through small pipes fastened along the sides of the large pipes, and having their lower ends curved so as to extend up into the latter. Their total lift was about 12 ft. and their capacity was about 40,000,000 gals. in 24 hours. The two pumps had a capacity of 10,000,000 gal., making a total of 50,000,000 gal. per 24 hours. In 20 days the area was unwatered and ready for excavation. The following season the balance of the area was inclosed and unwatered, making a total of 160 acres. Fig. 4 shows the character of the river bed.

The dams contained about 300,000 cu.yd. of material and were about 2¾ miles long, making the longest coffer-dam ever built, and within the limits of which was the largest area of any river bed ever exposed. The dams were built in a river with a four-mile current, and in from 9 to 18 ft. of water. C. H. Locher, of the contracting firm, was in direct charge of the building of the dams.

PLANT AND METHODS OF WORKING: SECTION 2

The power plant for the equipment used in carrying on the work on section No. 2 included two Rand air compressors (300 and 700 hp.). The rock broken down by blasting was removed by three Lidgerwood cableways having towers 110 ft. high and equipped with 3½-yd. skips. These skips were loaded by steam shovels and were dumped automatically, forming a wide spoil bank parallel with (and on the east side of) the channel, separating the channel from the river. Each cableway was operated by an engine having two cylinders 12½x15½ in., working by compressed air. The span of the cable-

ways was 726 ft. between centers of towers (including the 300-ft. channel and 300-ft. spoil bank). Later on, when it was decided to widen the channel in this

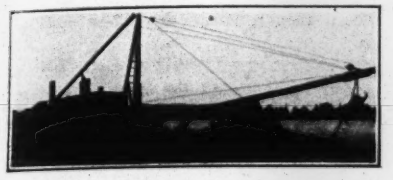


FIG. 3. UNLOADING MATERIAL FROM SCOWS BY A DERRICK BOAT WITH DRAG-LINE SCRAPER, TO FORM COFFER-DAM

"dry section" to 450 ft., the span was increased to 1200 ft. for two cableways and 1150 ft. for the third. The plant included also a blacksmith shop and machine shop.

The equipment for the work of excavation was divided into three separate units. Each unit consisted of a Marion steam shovel (No. 60), on traction wheels (with 2-yd. dipper), a cableway, drills, blasting outfit and necessary pumps. Three channeling machines were used on the work, the sides of the channel being channeled to the required depth. The entire equipment (with the exception of the steam shovel) was operated by compressed air. A narrow-gage locomotive was used for hauling supplies and repair parts from the shops to the works. Fig. 5 shows two of the cableways, with two steam shovels handling the broken rock at opposite ends of the section.

Excavation was started on section No. 2 in October, 1908, and progressed without delay until November, 1909, when it was necessary to close down for a period of five months, pending the decision to widen the channel an additional 150 ft. This widening has been done. The spoil bank from the additional excavation is on the west side of the channel, while that from the original 300 ft. cut is on the east side (Fig. 2).

DRILLING AND BLASTING

The drilling was done with tripod drills and later also by three traction drills, the development of C. H. Locher. The tripod drills were started with 3-in. steel and finished with 2-in. in the bottom. Mr. Locher conceived the idea that a drill made to work in a frame on land, in a similar manner to a marine drill, would be a success. He designed such a machine and had it built in the shops on the work. It showed increased drilling speed and a saving of time, as it was not necessary to change bits after each 2 ft. of hole drilled. Holes drilled with this machine were 4 in. diameter all the way down, and but one change in steel was made. After trying out this machine, two larger machines were built; one by the Ingersoll-Rand Co., and one by the Sullivan Machinery Co. The diameter of the holes drilled by these new machines was 5 in. all the way down. Holes were drilled with but one change in steel to 24 ft. depth, or 1 ft. below mean water level desired. The holes were spaced 8 ft. to 12 ft., depending on the depth of cut (8 to 18 ft). Three rows were drilled and loaded, and the field, comprising from 40 to 50 holes, was fired with excellent results.*

The principal explosive used was Red Cross low-freezing dynamite of both 40% and 60% strengths, manufactured by the Du Pont Powder Co. This was detonated with Victor regular and waterproof electric blasting caps. The conditions encountered proved that this powder can be used to advantage in very cold weather, as at various times the temperature fell below zero. In the summer the temperature was very high and the results obtained were uniform.

On section No. 2 the quantities moved above the 23-ft. depth approximated 704,-

FIG. 4. BED OF DETROIT RIVER EXPOSED BY UNWATERING THE CONTRACT SECTION, FOR EXCAVATION IN THE DRY

000 cu.yd. of rock and 180,000 cu.yd. of earth (bank measurement). Between the 23- and 24-ft. depths, 44,000 cu.yd. of rock and no earth. The area of section No. 1 which was sublet to Grant Smith & Co. and Locher was inclosed within the coffer-dam and excavation was carried on by the dry method to a depth of 23 ft. There were about 200,000 cu.yd. of rock in this section. The total channel length which was excavated in the dry was 5750 ft.

*These traction drills were described and illustrated in Engineering News, Dec. 2, 1909, and May 18, 1911.

WIDENING THE CHANNEL

The U. S. government decided in favor of the wider channel (450 ft. instead of 300 ft.) and under supplemental agreement the contractors (Grant Smith & Co. and Locher) provided for rock and earth excavation to widen the channel while the area was unwatered, and also for the removal of dams and material underlying the dams, to complete the original work required at sections Nos. 1 and 2. The price for the widening was $1.10 per cu.yd. for rock and 50c. for earth; for 100 ft. at the south end of section No. 2 (in the dry), $1.24 for rock and 60c. for earth. Removing material underlying the dam, $1.98 per cu.yd. at the north end; at the south end, $3.40, for rock and 60c. for earth; removing coffer-dam, $15,000. Total for work, $692,000. In all, about 1,500,000 cu.yd. of rock and 270,000 cu.yd. of earth were moved under the dry method. The complete 450-ft. channel is shown in Fig. 2.

Work was carried on in two shifts: 8 a.m. to 5 p.m., with one hour at noon for dinner; 6 p.m. to 3 a.m., with one hour at midnight for dinner. Work was stopped from Saturday midnight to Monday morning, as on all U. S. government contracts. At night acetylene lights were used. To keep the area unwatered, 13 pumps with a capacity of 50,000,000 gal. were installed, as water would seep in from the river and come up through the crevices and openings in the rock.

The excavation for this channel was carried on at an average of 55,000 cu.yd. of rock per month. On the west side of the channel, a dry wall was built 15 ft. high with a perpendicular face next to the cut. This was 18 ft. wide on the bottom, 5 ft. on the top, 1050 ft. long on the top, and 1090 ft. on the bottom, with dressed stone on the ends. Fig. 6 shows the channeled face of the solid rock, with the dry stone wall above it. On Dec. 20 the contractors finished the work of

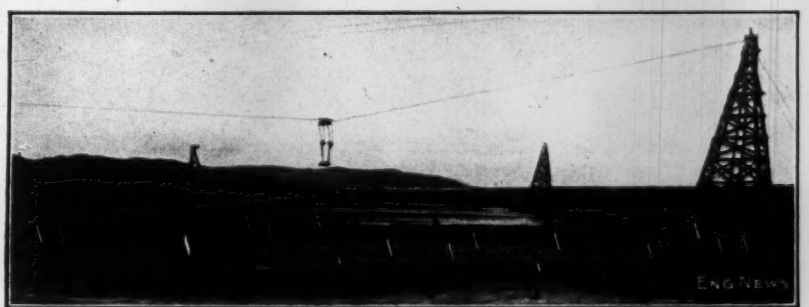


FIG. 5. EXCAVATING THE 450-FT. CHANNEL IN THE BED OF THE DETROIT RIVER

making the channel cut to the full width of 450 ft. Fig. 7 is a view of the completed portion. All pumps were stopped and the canal and the inclosed area within the dams was allowed to fill with water. The dams across the channel are being removed to full depth by subaqueous method, allowing for a slope of 1 on 1.

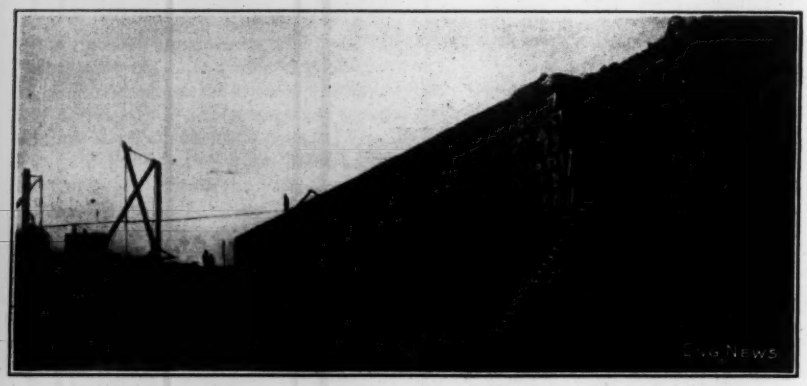


FIG. 6. THE WEST SIDE OF THE CHANNEL AT STONY ISLAND
(The face in the solid rock is cut by channeling machines. Above this is a dry stone retaining wall.)

In the execution of this contract, the contracting firm has broken all records for amount of work done. Notwithstanding complications over the construction of a permanent dam at the lower end of the cut, which made it necessary for the plant to stop all work for several months, the work of excavating the channel to a full width of 450 ft. was accomplished within the time specified on the original project for 300 ft. Fig. 5 shows the south end of the channel (looking north).

CAMP

The contractors for section No. 2 leased Stony Island for their camp, which was an ideal one. A school was built and the children on the island were taught by a teacher appointed by the local authorities. On Sunday this building served as a church. At least once every two weeks, and sometimes every week, the contractors furnished music for dances and other amusements for the families of the men. The camp was remarkably free from sickness, which can be accounted for by the sanitary conditions enforced, and the fact that pure drinking water was furnished. Pipe lines were laid from the compressor plant to all parts of the camp and distilled water (condensed steam) was supplied. In the summer, ice was furnished for drinking water and other purposes. These pipe lines were also available in case of fire. The company owned its power boats and all material and supplies were delivered to the mainland and carried to their work. The U. S. government maintained a customs officer on the island to pass on any supplies imported through Canada. G. P. Locher was superintendent in charge for the contractors.

SECTION NO. 3

Work on section No. 3 is entirely subaqueous. The rock was drilled and blasted, and then removed by dredging. The quantities in this section were as follows: to 22 ft. depth, 555,200 cu.yd. of rock; from the 22 ft. to the 24 ft. depth, 316,180 cu.yd.; total, 871,380 cu.yd. The section is 18,250 ft. long and 300 ft. wide, with side slopes of 1 on 1.

The contract for this section was sublet in three parallel sections of 100 ft. in width, extending for the full length (Fig. 1). Contracts were sublet to M. Sullivan for the eastern section; Dunbar & Sullivan Dredging Co. for the middle section, and the Buffalo Dredging Co. for the western section. The conditions and methods of working are practically the same on the three parallel sections. The superintendents in charge for the three contracting firms are, respectively, John Sullivan, F. E. Dunbar and A. H. Goode. It is expected that this section will be finished and ready for navigation about July 1, 1912.

SUBMARINE DRILLING AND BLASTING

For its work on the west side of section No. 3 the Buffalo Dredging Co. assigned (with other equipment) its drill boat No. 5, which was designed by William Logan, the company's general superintendent. It is of steel construction and represents the most modern methods in drill-boat construction. It is equipped with five drills. The length of the boat is 138 ft. on deck and 130 ft. on bottom; beam, 31 ft. 3 in.; depth, 6 ft. 4 in. The house is of steel, 110x19 ft. 2 in.; its height on drill side is 12½ ft., sloping down to 11 ft. on the back. The roof overlaps side walls about 3 ft. In the hull there are four transverse bulkheads and four longitudinal bulkheads, dividing the hull into 25 compartments, 6 ft. 3 in.x 26 ft. The compartments can be entered through manholes from the deck. Steam siphons are used to keep the hull free from water.*

Steam is used for lifting the drills and moving the drill frames along the deck. The frames move along the deck on three tracks of 45-lb. rails. Each frame is equipped with a steam engine for hoisting the drill and moving the frame and is independent of all others. The drills are raised and lowered on their frames by steam, effecting a great saving in time as against slow-moving hydraulic machines. The drill runner operates a foot brake on the same shaft as the hoisting drum, and with this he can get a feed at any speed desired. On the same shaft with the drum and foot brake and inside the bearings, is a notched wheel, into the notches of which a pawl may be slipped and the drill held in any position. Moving the drill along the deck is done by the steam engine which operates the hoist. Each frame has a heavy chain fastened to the deck at each end, just a little beyond the limit of its deck range.

The coal bunker is of steel and has a

FIG. 7. THE COMPLETED PORTION OF DRY EXCAVATION FOR 450-FT. CHANNEL

*A very similar drill boat used on the Livingstone Channel work by the Great Lakes Dredge & Dock Co., was illustrated and described in our issue of Dec. 17, 1908.—Editor.

capacity of 100 tons. This is more than one week's supply, as the boiler (marine type, 12½x9 ft.) burns 10 tons of coal in 24 hours. The boat is provided with a steam hammer, steam capstans for moving back and forth over the ranges, and steam spud engines. The blacksmith shop is equipped with a crane for handling the drill steel. The boat is also equipped with an acetylene-gas plant.

In places the rock drilled was covered

FIG. 8. BLAST OF A FIELD OF DRILL HOLES IN THE DETROIT RIVER (FOR SECTION 3 OF THE LIVINGSTON CHANNEL)

with mud or silt. In order to prevent this from running into the bore hole and causing trouble and loss of time, Mr. Logan designed an apparatus called the "mud pipe." This is a telescopic casing 8 in. diameter, which is attached to the drill frame and can be lowered through the mud to the rock. In the casing, but above the level of the mud, are two openings. The drill is worked in this casing, and as the drill works up and down cutting the rock, the cuttings are carried out by the river current through the openings in the pipe and the bore hole is kept clean. After the hole has been cleaned it is loaded before the "mud pipe" is raised.

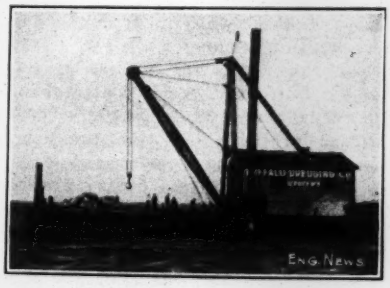


FIG. 9. DERRICK BOAT USED BY DIVERS FOR INSPECTING AND REMOVING ROCK FROM DREDGED CHANNEL

The contractors move their drill boats over sections on ranges determined by the U. S. Engineers, and the distance moved each time is measured on two wires stretched across the cut from the boat to stakes which interpolate the range stakes.

Holes are spaced and drilled 5 ft. apart longitudinally and 4 ft. laterally. On this work blasting has been done principally in field shooting, although single holes and single ranges have been fired. A single line of holes fired at one time is known as a range; a multiple row of holes as field shooting. The number of holes drilled by a five-drill boat in moving across a 100-ft. section is about 615, all of which have been fired in a single field. Fig. 8 shows a field blast, using the Du Pont Powder Co.'s Atlas 60% dynamite.

In the summer, when the water was warm, blasting was done in fields. In the early spring and last fall when the water was colder, ranges and single holes were blasted, as the powder used became chilled and insensitive from exposure to cold water.

Loading or charging the hole is done by the use of a "loading gun," with a barrel 2 in. diameter. The Atlas brand of dynamite was used (60% strength), packed in cartridges 2 in. diameter so that they fit the barrel as they are loaded into the lower end. The barrel is slit in order to allow the wires from the Victor electric blasting caps to pass through as the charge is loaded into the bore hole. A piston or plunger passes through the top to force the charge of dynamite into place in the bore hole in the rock. A chain is fastened to the top of the barrel and passes up through a block at the top of the drill frame and falls to the deck. After the barrel is charged it is lowered through the "mud pipe" into the bore hole and when the bottom of the barrel reaches the bottom of the hole, three men raise the barrel by the chain from the hole, and the piston holds the explosive charge in place. Should the depth of the hole require a greater charge than the capacity of the barrel, the operation is repeated.

The Buffalo Dredging Co.'s dredge boat No. 7, used on this work, has a displacement of 1000 tons; it is equipped with engines of 500 hp., and has a 6-yd. dipper. The dredge boats load the rock into dump scows of 500 to 600 cu.yd. capacity. After being loaded, these scows are towed to appointed dumping grounds and dumped. The men work in two shifts of 10 hours in the day and 12 hours at night; the men change shifts each week. After the dredge boats, the derrick boat (Fig. 9) with complete diving outfit passes over the section. The function of this boat is to remove to the clear depth of water desired all blasted rock which has been left above the grade line by the dredge.

MARKING AND REGULATION

In order to mark the channel through this dry section (No. 2) the U. S. government has built concrete piers for six lights (three lights on each side). Four more lights will be placed along the subaqueous section marking the channel to the lake. The foundations for the four lights for the subaqueous section were built on the dry section in the form of cribs. They will be floated down the river and sunk to place to form the bases for the permanent lights.

Between the lower end of the "dry section" and the upper end of Bois Blanc Island there is a strong cross-current, and as a protection to boats passing down a permanent dam will probably be built between these points. As a further necessary improvement the width of the channel in all subaqueous work should be widened from 300 ft. to 450 ft., which is the width through the "dry section."

Water Supply For Fire Protection in Mines*

Water supply for surface works and water supply for underground works are dissimilar problems, and should be separately provided for and not be interconnected. For underground works, water supply should be ample and preferably stored underground, thus avoiding trouble from hydraulic head, signaling service or freezing. Water for the automatic sprinklers proper may be very limited in quantity if stored in a tank under high pressure. For limited spaces such as storerooms, stables, etc., there is no record of fires which 3000 gal. of water under pressure have failed to extinguish. Including the necessary air, a pressure tank of 4500 to 6000 gal. should be adequate. This, applied to local pieces of sprinkler equipment as for storeroom or stable, would call for space ranging from 5000 to 10,000 ft. area.

For pump supply which may be called upon to supply hose streams for long periods, the quantity of water is a different matter. As much as 250,000 gal., which amount may be stored in a relatively small space, will run a number of hose streams as heavy as can be handled by one man for a number of hours; for example, 125 gal. per hose stream per minute would mean a ⅞-in. nozzle for 2000 minutes or 10 such streams for more than three hours.

As to the relative efficiency of various pumps; possibly a centrifugal pump taking water under head and operated by induction motor will give the better satisfaction, because it is easiest operated by unskilled men, and may be put in service from a distance by signal wire or telephone. Study should be made of quantity of water required, size and construction of pipes, nozzles and styles of fitting. For working in confined space of a mine in presence of heat and gases, heavy hose streams would seem impracticable, probably nothing in excess of

*Extracts from the first report of the Committee on Mine Fires (H. M. Wilson, Pittsburg, Chairman); presented at the annual meeting of the National Fire Protection Association at Chicago, May 14 to 16. The report outlined investigations to be made as to fire prevention and protection of underground workings, shafts and surface plant.

1¼ to 1½ in. taken off a 2-in. line, or from long distance from source of supply, 3- or 4-in. line, as indicated by distance and friction head.

Commercial Results from the Improvement of a Tidal Creek for Navigation

By R. R. Raymond*

From Delaware City to Lewes, Delaware, the shores of Delaware River and Bay possess characteristics which tend to discourage water-borne commerce. The water is generally shallow, the fore-shore consists largely of shifting sand, and the "fast" land, as it is known locally, is generally several miles inland from the shore line proper.

Between the "fast" land and the shore line lie great marshes averaging several miles in width and rarely rising much above tide level. The drainage of such marshes is accomplished naturally by ditches scoured out by the water, uniting into channels and finally into navigable waterways maintained by the tides.

Man has assisted the drainage in many places by digging ditches as straight as possible across the meadows to the nearest channels, thus affording more direct escape for the surface water and at the same time subdividing the land into areas suitable for cultivation when the salt water can be excluded, or for harvesting the natural growth of marsh hay.

The larger channels are remarkably permanent in location and possess depth enough for navigation by small boats, but they are very tortuous and consequently difficult to travel for boats of sufficient size to carry cargoes, and the actual distance traversed between two points on a stream is often many times the distance in a bee line.

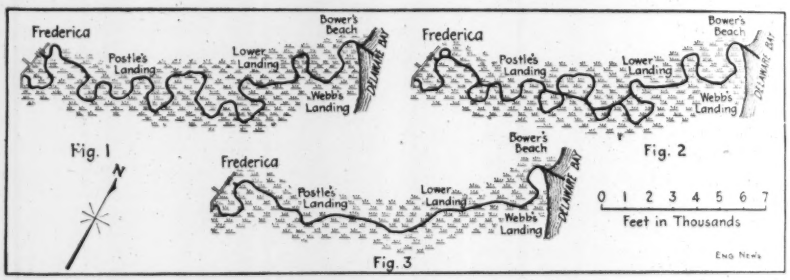
Although settled early in the 17th century, the country bordering the marshes, except where reached by railroad, has remained until recent years in a somewhat primitive condition. Blessed by nature with a fertile soil and a good climate, farm lands capable of yielding untold wealth have lain idle and unproductive because no reasonably economical way was available to transport products to market.

Except for a few desultory efforts to relieve the difficulties in some of the channels, no serious work of sufficient scope to affect the real situation was undertaken until recent years, principally within the last decade; and so for nearly three centuries the territory under consideration has awaited the touch necessary to cause it to spring into life and to yield in abundance the fruits of its great resources.

The short section of coast mentioned, about 65 miles in length, has some ten channels of commercial importance. Of these we may select one as a type, in order to point out some interesting features of the engineering problems presented and the physical and commercial results obtained.

The Murderkill River will serve as an example. It has been under improvement longer than most of the other streams, but this serves to make more complete the lessons it teaches. It lies in Kent County and empties into Delaware Bay about 25 miles above Cape Henlopen and 80 miles below Philadelphia.

Originally its navigable portion was about 13 miles long and extremely crooked (see Fig. 1), traversing marshes or meadows and touching "fast" land at only a few widely separated points. The head of navigation is seven miles from the nearest railroad, to which the sandy soil makes hauling by wagon difficult and costly. The excellent farming country surrounding the town of Frederica near the head of navigation is therefore almost wholly dependent on the river for transportation to suitable markets. At the mouth of the river is a summer resort of considerable local importance, known as Bower's Beach, which is the headquarters of the fish and clam industry of this section.



Map Showing Various Stages in the Improvement of the Murderkill River, Delaware

Many years ago, efforts were made by private parties to improve the river by cutting canals across the marshes to avoid some of the most difficult bends, and by providing a channel across the flats at the mouth of the river. These efforts were continued from time to time, causing a progressive betterment of conditions. Even 30 years ago a commerce estimated to be worth about a quarter of a million dollars a year existed.

In 1892 the United States Government assumed the duty of improving and maintaining the channel, but funds were supplied in small appropriations so that the improvement has been gradual and has only recently reached a stage which enables a steamboat to run regularly between Philadelphia and Frederica without regard to the tides. From now on the full benefit of the long years of labor and effort should be derived, and already the commerce has reached a value of a million and a half dollars annually.

We may pass over the improvement of the mouth of the river by stating briefly that it involves dredging and maintaining a channel across the flats. A jetty should be constructed, but at present the bank of material dredged each year serves as a jetty until leveled by wave and tide, and the cost of annual redredging, being less than 3% of the estimated cost of a jetty, does not warrant the construction of the latter under existing conditions. The improvement of the river itself consists in removing the shoals, cutting back sharp points and, most important of all, making cutoffs.

Dredging the soft materials of the marsh is easy and comparatively cheap. The cheapest method is by means of a hydraulic dredge and this method has been employed to some extent; but as the dredged material must be prevented from returning to the channel, hydraulic dredging generally involves the construction of banks or levees. The plant usually employed heretofore has been a clam-shell dredge with a very long boom, which deposits the material well back on the marsh so that the foot of the near slope is at least 10 ft. from the river. As dredging progresses, the banks of the river rise above the neighboring level of the marsh and in due course of time it will be possible to adopt the hydraulic dredge to pump over the banks now formed or forming.

Even the clam-shell dredge demands skill and judgment in its operation, for if the marsh be overloaded the new bank will settle, sometimes sloughing off into the river more material than was taken out, and sometimes sinking vertically. In the latter case, lateral pressure may cause the bed of the river to rise above the water surface, an action which is increased sometimes by the expansion of the highly compressible material when relieved of the weight of mud formerly pressing it down. We have thus presented the curious phenomenon of a reduction of depth due to the removal of material.

Although not connected with the improvement of this river, it may be interesting to note that under certain condi-

*Major, Corps of Engineers, U. S. A., Wilmington, Del.

tions the depth might be increased by dumping material into the channel, as was shown by an attempt to build a road across a marsh similar to that through which the Murderkill River flows. A fill of gravel, usually excellent for an embankment, so compressed the marsh as to produce a canal instead of a road, and great expense was incurred in correcting the mishap.

From Fig. 1, which shows the original course of the river, it will be seen that the 13 miles of channel presented many difficult turns. To eliminate these and at the same time shorten the route, six canals were dug by private enterprise, which were located where primitive methods and lack of funds demanded, generally without proper regard for the flow of the tides. This work reduced the length of the route by a little more than 4 miles, but did not fully rectify the river even where cutoffs were made, and the effect of improperly located cutoffs was frequently to produce eddies which caused shoaling.

Fig. 2 shows the general effect of the work done prior to the beginning of Government work. The project adopted by the Government in 1892 provided for a low-water channel 7 ft. deep and 80 ft. wide at the bottom from Frederica to Delaware Bay. The full width has not been secured, but one cutoff has been made near Lower Landing, the old cutoffs have been corrected as far as possible, and the channel throughout has been made uniform and free from irregularities which might cause shoaling.

Fig. 3 shows the condition which the river is approaching and which it has actually reached except for one cutoff west of Webb's Landing and another south of Postle's Landing, which yet remain to be dug. The old bends have been omitted from the map. They still exist but gradually fill up, some of those earliest cut off being now impassable for small boats. The river has been shortened to 8.7 miles and the remaining work will further shorten it to 7.8 miles, or about 55% of its original length.

The result of the work done has been to provide free navigation for steam and gasoline cargo carriers of convenient and economical size. Not only is the river route shortened, but most of the difficult bends have been eliminated. It was formerly common for a steamer to drive her nose deep into the soft bank and lose much time in extricating herself, so that snubbing posts were used to assist boats in making the turns. Saving time is always important, even where it does not enable more trips to be made in a given period; but the chief value of the improvement from the view point of navigation lies in the possibility of using larger and more economical carriers and operating them regularly.

The methods employed have, however, other valuable results. The shortening of the channel by more than 5 miles has reduced the labor and cost of maintenance in an even greater ratio. Shoaling still takes place in the bends but is no longer of any importance, except to force more water through the cutoffs and so improve them and assist in maintaining them. Steamers rarely collide with the banks to push masses of mud into the channel.

Important changes in tidal flow have taken place. The average range of tide at the mouth of the river is about 5 ft., while at the head of navigation under original conditions the range was 1 ft. The construction of cutoffs has largely increased the slope by diminishing the horizontal distance between the upper river and the mouth and has produced a more rapid flow of the tide, which assists in maintaining the channel and at the same time the cutoffs enlarge the tidal prism. The freer movements of the water in their turn increase the range of the tides at the head waters of the river so that today the range is 3 ft. The net result is a greatly increased volume of tidal flow and more effective scouring velocities. Thus the cutoffs are practically always self-maintaining.

The increased tidal range in the upper river has also had a beneficial effect on the drainage of adjacent low land, enabling tracts, formerly unproductive, to yield crops and increase in value. A marked improvement in the health of the community has been observed since the drainage has been effective.

The final effect of the simple improvements made in this and other similar rivers of this region is a rapid development of the country. Food products are poured into the markets of Philadelphia and other cities. Canneries spring up and new farms are established to supply the corn, tomatoes and other vegetables and fruits for canning. Fertilizer factories are built to supply the farms. Various industries develop. Stores are opened to provide the products of other states and interstate commerce grows and flourishes. Land increases in value.

In view of the benefits obtained, the cost of the improvements seems slight. On the Murderkill River only $72,000 has been expended in 20 years, a sum equal to considerably less than ⅓ of 1% of the value of the commerce benefited.

Letters to the Editor

A Request for Laboratory Methods of Testing Sand

Sir—The National Association of Cement Users, through its Committee on Specifications and Methods of Tests for Concrete Materials, is investigating the problem of testing sand and other aggregates for use in portland cement mortar and concrete to determine their suitability for such use. This committee desires to get into communication with all laboratories who are now testing sand and other aggregates, and to secure their coöperation with the committee in formulating standard tests for these materials. The committee is particularly anxious to secure samples of sand which appear suitable for use in concrete, but in practice prove unsuited. The committee would welcome communications from all interested in the subject who are willing to coöperate in this work to the extent of furnishing data obtained in their laboratory or field practice, or of recommending forms of tests which may be considered by the committee. Communications should be addressed to Cloyd M. Chapman, Secretary, care of Westinghouse, Church, Kerr & Co., 10 Bridge St., New York.

NATIONAL ASSOCIATION OF CEMENT USERS.
COMMITTEE ON SPECIFICATIONS
AND METHODS OF TESTS
FOR CONCRETE MATERIALS.

New York, May 14, 1912.

Wood-Stave Pipe and Concrete Pipe

Sir—Relative to the letter of Winfield Holbrook in your issue of Apr. 25, 1912, p. 799, in which he discusses some points on wood-stave pipe and concrete pipe suggested by my article on "Description of the Prosser Division of the Sunnyside Unit, Yakima Project, U. S. Reclamation Service":

Mr. Holbrook's statement that the head at the ends of the wood pipe is too light to cause a penetration (meaning, I assume, saturation of the wood) of over ½ in. from the inside is interesting, and if he has any data in proof of this statement it would be well to present them.

The static head on the wood at both ends is 20 ft., and it was considered that this is sufficient to produce saturation of the 2-in. vertical-grain staves used in this pipe.

The writer would have preferred the use of 1½-in. staves, but on account of the rock cuts and heavy gravel, interspersed with boulders, through which the pipe passes it seemed desirable to use very rigid construction to prevent the pipe from becoming distorted.

A more careful reading of the article will clear Mr. Holbrook's doubts regarding the determination of the size of the pipes. It was not assumed that the 30½-in. concrete pipe had carrying capacity equal to the 31-in. wood-stave pipe. The calculated loss of head per 1000 ft. for the concrete pipe is 3.86 ft., for the wood pipe 2.85 ft. The former is based on the use of Kutter's $n = 0.013$, and the latter is based on a special formula determined from experiments. The writer has recently published the results of some experiments* which would indicate that the loss in the wood pipe is even smaller than given above. He also believes that experiments would show that the loss calculated for the concrete pipe is excessive, as it is not likely that there is such a large difference in the carrying capacity of wood and concrete pipe. There is a lamentable dearth of experiments on the carrying capacity of concrete pipes and channels.

E. A. MORITZ.

North Yakima, Wash.,
May 8, 1912.

*Trans. Am. Soc. C. E., Vol. LXXIV, p. 411.

Re-rolled Rails for Concrete Reinforcement

Sir—In your issue of Apr. 4, 1912, p. 651, the "note of warning" regarding the use of re-rolled rails for concrete reinforcement sounded by E. W. Stern in a previous issue, is further discussed by Prof. Ira H. Woolson.

The results of Prof. Woolson's experience in testing rail-steel reinforcing bar is so much at variance with those the writer has noted that it would interest me to obtain full data on his tests. How many tests were made the basis of his conclusions? From how many sources did the specimens originate? How were bending tests made? Were the loads applied suddenly or gradually and what was the speed of the testing machine in making the tensile tests?

It is a well known fact, admitted by recognized authorities, that the results obtained in testing depend upon the methods used. If, for instance, the speed of the testing machine is 3 in. or more per minute, the results will vary greatly in elasticity, elongation, reduction, and in the appearance of the fracture, from the results which will be obtained if the speed of the machine is 1 in. or less in three minutes.

Steel when slowly fractured will present a silky fibrous appearance, while the same steel when disrupted suddenly will show a distinctly granular fracture. The writer has often seen material rejected for granular fracture when this appearance was due, not to the quality of the steel, but solely to faulty practice in conducting tests.

The writer's experience in testing has demonstrated that with either rail steel or hard or soft billet steel, the sudden application of tensile stress will establish a pronounced decrease in ultimate strength and elongation. In a series of tests on hard grade steel an ultimate resistance of 84,700 lb. per sq.in., and an elongation of 12.5% was noted when the stresses were applied suddenly; while an ultimate strength of 96,300 lb. per sq.in., and an elongation of 17.3% was shown when the same steel was stressed gradually. The same degree of variation prevailed on soft and medium steel and on bars of various gages. It is very possible that Mr. Woolson's condemnation of rail steel is based on erroneous conclusions drawn from similar test experience based on rail steel only.

From many hundreds of tests made on this material the writer should say that rail steel is rather more uniform than other grades. There is abundant reason for this. Whether openhearth or bessemer, rails are made from carefully selected stock. Careful analysis is made from the test ingot, and rechecked by drillings from the finished rails, thereby insuring that results are uniformly accurate according to mill reports. The requirement of a 15% or 20% discard from the ingot eliminates as far as is practically possible the effect of segregation from the finished material. Drop tests are made by the impact of a 2000-lb. tup from heights of 15 to 18 ft., according to the section under test. This impact test is a practical test to demonstrate the ductility and quality of the material.

The character of service required of a rail, being more severe than that imposed on any other grade of steel, should, after all, remain its best recommendation as regards its fitness for any use entailing stress and shock of any kind. If would seem to the writer that engineers might better concern themselves with the possible quality of other grades of reinforcing, of whose composition we know very little, and surrounding whose manufacture few, if any, restrictions are thrown. The quality of rail-steel bar is a known proven factor, which cannot be said of many other grades now largely used.

W. H. WOODCOCK.

Buffalo, N. Y., May 6, 1912.

[This letter was submitted to Prof. Woolson and he has sent the following reply.—ED.]

Sir—I have read the letter of W. H. Woodcock, of Buffalo, who writes in further defense of re-rolled steel rails for reinforcing metal in concrete construction, and asks information regarding number and methods of tests, also sources of material to which I referred in my communication on the same subject. In reply, I would say that my statements were based upon experience obtained from testing commercial stock which was sent to my laboratory for test by interested parties. These tests extended over a period of some years from the time this material first appeared upon the market.

Mr. Woodcock's statements as to the effect of slow and very rapid application of test load upon the character of fractures is a very well known phenomenon, and no person with experience in such work would think of condemning a piece of steel because it gave a crystalline fracture and low ductility when broken by shock or a speed approaching same.

The tensile tests were made at a speed of approximately 1 in. per minute, which is entirely favorable to the test piece.

Most of the bending tests were made in two operations, the first bending to a round-angled V shape, and the second closing down to the desired position. This is generally considered more favorable to the test specimen than when the work is done in one operation, since a period of rest is introduced. Later tests made upon a special bending machine were confirmatory of previous results. Usually both tensile and bending tests were made on the same lot of samples, one serving to check the other as to quality of the metal.

One set of re-rolled ½-in. twisted bars which were sent in for test by a prominent builder some years ago is well remembered. They were so bad they were easily broken by clamping one end of a 20-in. bar in a vise and pulling sidewise on the other end with one hand. The specimens broke easily before they had bent 30°. This was my very worst experience.

I have not the records of these tests at hand now, nor the time to go and look them up to ascertain from how many different sources the test bars came. As a matter of fact, I cannot see that it is a material point in the controversy. It was commercial stock on the market for a specific purpose. That is all that interests the consumer.

As stated in my previous letter, I do not condemn all re-rolled reinforcing bars; it is simply that my experience has shown them to be unreliable in quality.

IRA H. WOOLSON,
Consulting Engineer.

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*Illustrated.

Engineering News

ENGINEERING NEWS has often criticized the expenditure of public funds on waterway improvements which are not justified on economic considerations. We are equally ready, however, to commend waterway development where the conditions warrant the expenditure. Elsewhere in this issue there is described a modest piece of river-improvement work carried on by the federal government which is typical of a considerable amount of waterway improvement by the government and which deserves cordial praise.

The humorists of the daily press have for years portrayed the River and Harbor appropriations for creeks with unpronounceable names as an example of the waste of public funds. It is probable, however, that of all the money spent by the government for the benefit of water transportation, none is more usefully spent than that devoted to the small tidal streams along the coast. These streams reach districts which have often no other means of transport than the water route. Often, as in the case of the work described in this issue, not very much expenditure is necessary on improvement of the river to enable light-draft steamers to run on these rivers and make regular trips. The opening of these minor waterways is made even more advantageous by the advent of the motor boat, which can operate on light traffic at much less cost than a steamer.

Rapid Evaporation in a Locomotive Firebox

In another part of this issue we have noted a preliminary report of comparative tests on two locomotive boilers with different types of firebox construction. In order to compare the action in the two firebox sections uninfluenced by the effect of the barrels, the natural expedient was adopted of extending the back tube sheet to serve as a complete diaphragm between the barrel and firebox sections. The results of the tests on the separate sections of these boilers are very interesting and may have considerable influence on boiler design. The tests at present are of only minor interest in their comparison of small differences in the results of the two types of firebox construction. The performance in each case seems to have been substantially the same so that the comparison of actions in barrel and firebox sections is strengthened by the duplication of tests.

It was found that about 50% of the work done by the entire boiler was secured from 6% of the heating surface —that in the firebox section. The evaporation here mounted up to the surprising figure of 50 lb. per sq.ft. per hour. This performance should be compared with the 3 lb. maximum allowed in stationary practice and the 15 to 20 lb. which have been noted in heavily forced locomotive boilers and the 20 lb. of Prof. Bone's gas-fired flameless-combustion radiation-transfer steam generator described in ENGINEERING NEWS, Jan. 18, 1912.

The first impression as to this extreme rate of evaporation per unit area of heating surface is that the rate of discharge of steam off the boiler plate and up through the water legs of the firebox section is going to lead to difficulty in operation. A little study of this point then is helpful. A moderate pressure for these locomotive boilers to carry is 175 lb. gage and at this pressure 1 lb. of steam has a volume of only 2.41 cu.ft. Therefore, the discharge of steam from a square foot of firebox heating surface corresponding to an evaporation of 50 lb. per hour may be expressed as 2 cu.ft. per minute. This would give a linear velocity of steam away from the plate, of only 2 ft. per minute. With a water-leg 4 in. wide and 6 ft. high, as is common in such boilers, 12 cu.ft. of steam per minute would have to pass an opening of ⅓ sq.ft. cross-section. This would lead to a velocity of 36 ft. per minute. This very moderate velocity should not give any trouble in keeping the water-leg supplied with water, even though the downward current of water and the upward current of steam are opposed.

Defective Sands for Concrete Aggregate

It is now several years since the more advanced engineers responsible for concrete work concluded that the quality of the sand, which in good concrete makes up about 30% of the bulk of the mass, was not being sufficiently guarded. Extensive and thorough tests had been demanded of all the cement entering into acceptable concrete, but the fine aggregate was passed upon assurance of its being clean and sharp. The method of determining this cleanness and sharpness was ordinarily purely ocular or manual and depended entirely upon the personal equation of the tester. Usually he merely rubbed together a handful or two of the sand and determined its quality by the feeling or by the amount of dirt left on the palm. Very careful engineers required a sedimentary percentage test in a graduated glass, rejecting such sand as showed an excessive deposit of silt or

soft material after a night's settling. But here again the process was only approximate, the amount justifying rejection being a matter of personal opinion.

The first step toward change in this old and inaccurate method of selecting sand was the granularmetric analysis first proposed in this country, we believe, by Wm. B. Fuller. This test provided an optimum gradation in the size of the sand particles, the more closely to which the gradation of an actual sand approached, the more thoroughly would that sand fill the voids in the mass to produce a denser and consequently a stronger concrete. Here was a definite specification which could be followed in a uniform manner and which, when rightly observed, tended toward a better concrete. Some time later, analyses of the mineral constituents of the sand were also proposed and occasionally followed, though their application was somewhat too involved for ordinary laboratory work and their value not assured.

Meanwhile a number of engineers were encountering sands which successfully passed the elementary tests for sharpness and cleanness and which possessed a highly efficient gradation curve but which in mortar tests or in actual work failed to develop a strength at all comparable with that developed by standard testing sands or with the safe strengths required in construction. One of the first references to these conditions was made by Sanford E. Thompson in a discussion on "Impurities in Sand for Concrete" in the *Transactions* of the American Society of Civil Engineers, Vol. 65, 1909. As a result of his studies of the question, Mr. Thompson concluded that the loss in strength was due to the presence of silt in the sand, which silt, made up of vegetable organisms, was not in sufficient quantity to be detected by ordinary tests, but nevertheless acted to restrict the setting qualities of the cement. It is to be noted that neither Mr. Thompson nor any of those who took part in the discussion could furnish any explanation why from 1% to 2% of vegetable silt should so seriously affect the cement-setting process. It was merely shown that it did and that the only safeguard was a thorough washing of the sand when under a mortar test it seemed seriously to affect the strength of a concrete made from it and other materials which passed their own tests satisfactorily.

About this time a tendency to stiffen the sand test was shown in the following clause from the Report of the Joint Committee on Concrete and Reinforced Concrete:

> Mortars composed of one part of portland cement and three parts fine aggregate by weight, when made into briquettes should show a tensile strength of at least 70% of the strength of 1:3 mortar of the same consistency made with the same cement and standard Ottawa sand.

This clause, if inserted in concrete specifications, would exclude many defective sands even though it does not help in the question of what makes them defective. Unfortunately, it is rarely called into play. There are a number of reasons for this, principal among which is the reluctance of engineers to force the contractor to make what he considers an unusual and unnecessary test. There are also the difficulties of sampling and storage to overcome, but in the main the trouble lies in the inertia of established custom. Attempts are being made by several societies to bring this matter of definite sand specifications to a head and we can expect a marked advance in the near future. The letter on p. 1043 of this issue, from a committee of the National Association of Cement Users, indicates a desirable activity which is hopeful. As an aid to that committee's work and to further the study of this question of defective sands, we trust that all engineers will read and ponder the contributions to the subject by Messrs. Freeman, Free and Gaines on p. 1022 of this issue.

In the beginning it will be noted that Mr. Freeman's researches did not promise more than the earlier ones reported, as described above, by Mr. Thompson. There was the same apparently excellent sand with the same unreliability as to strength, an unreliability due apparently to a small quantity of deleterious matter which could be removed by a thorough washing. There are from this point on, however, two marked differences from previous experience. The ordinary assumptions that the deleterious matter was a vegetable organism seemed hardly to hold, on account of the location of the sandpit—and, further, the sand set up well with one brand of good portland cement and failed entirely to combine with half a dozen other equally good cements.

In regard to this first difference it is true that the earlier assumption that the harmful silt was a vegetable organism had not particularly aided the study of the nature of the reactions involved. No one has intelligently explained why 1% of vegetable organism should prevent portland cement from setting. But the basis of all the theorizing has been on this very assumption, so it is of advantage to know that there may be deleterious substances other than vegetable organisms. The difference in behavior of the various cements is much more important, however. That indicates a physical or chemical reaction involving the complex nature of portland cement and immediately throws the problem into the realms of the higher and specialized sciences. As tentative explanations of these reactions we append to Mr. Freeman's article papers by two chemical experts. We cast no reflections on the ability of the average engineer when we say that these explanations are far beyond his capacity or even his interest, but even the most practical of engineers recognizes the necessity for such scientific studies of those problems which cannot be solved by the ordinary simple methods.

It is to be hoped that the investigations proposed can be carried out, if not by the proposed contributions from interested parties, at least by some of the governmental or institutional laboratories which are equipped and manned for just such work. We especially emphasize Mr. Freeman's idea that engineers who have had experiences similar to the ones noted here should send to us brief reports thereof so that we may put them on record for the benefit of the investigators in particular and the profession in general. When there has been collected enough testimony to show that the conditions are general and not sporadic, it will be time enough to discuss means for supplying the funds to carry on the proposed studies.

We are quite aware that these defective sands are the exception rather than the rule and that there will be a number of engineers and contractors who will think the whole investigation a highly interesting laboratory amusement of no value whatsoever to practical construction. The fact remains that there are sands whose availability for concrete aggregate cannot be predicted under any of the ordinary methods now in vogue, and that every so often one of these sands gets into a structure to its future detriment. The reasonable thing to do then is to provide some practicable test for such sands, and to devise such a test intelligently requires a knowledge of the reasons for the defects in the sand. If the articles in this issue of ENGINEERING NEWS aid in the determination of these reasons they will have served their purpose.

Comments on Sewer Assessments Suggested by a Duluth Court Decision

About the only universal scientifically grounded rule governing assessments to meet the cost of such municipal improvements as paving and sewerage is that the assessments must be proportionate to the benefits received. In determining how this shall be done those who make the assessments generally follow long established local precedent or else what they conceive to be the line of least resistance. The most common plan is to assume that the benefit is directly proportional to the street frontage. Next to this probably comes the land-area basis. Sometimes the two are combined. There are various modifications to cover corner lots, extra deep lots and other conditions which tend to make assessments obvi-

ously disproportionate to benefits if the strict rule be followed.

Two factors which are either overlooked entirely or else not given sufficient weight, and which in any event are rarely figured on a scientific basis, are (1) the division of benefit into public and private, or that pertaining to the city as a whole and to specific property or individuals, and (2) the division of private benefit into (a) availability and (b) actual use of the specific improvement in question.

The most able and complete discussion of this subject of which we know is to be found, if one is so lucky as to be able to gain access to it, in a report on assessments for the sewerage system of Brockton, Mass., which system included a pumping station and filter beds. The report was made by F. Herbert Snow, then city engineer of Brockton, but for a number of years past chief engineer of the Pennsylvania Department of Health. The report was noted briefly on p. 51 and outlined and discussed editorially on pp. 53-4 of our issue of July 19, 1894. Since the issue of ENGINEERING NEWS just named will be no more accessible to a large percentage of our readers than is Mr. Snow's report, we reprint herewith Mr. Snow's own summary of recommendations and a portion of our comments on the plan. First come the recommendations, as follows:

It is recommended that one-fourth of the total cost of the sewerage system be raised by first assessment, one-half by rental, and the remainder by general tax. It is also recommended that first assessment be based on area and frontage of land adjacent to sewers—0.6 on area within 125 ft. of the first street line and 0.4 on frontage; that the first assessment be collected in one payment and credited to construction account; that the unit of rental be 1000 gal. of water reaching the sewer, this to be ascertained from meter gagings of the water department, and to be corrected for water finding another outlet than the sewer by a system of discounts; 70% to be deducted from water supply of shops and 20% from water supply of houses having sill-cocks.

And it is further recommended that abuttors be not compelled to enter the sewer as soon as completed; that no one be allowed to enter without a permit; that no rent be charged users before Jan. 1, 1895, rents starting from that date; that such rents be charged from the first of the month following that in which the permit is dated; and that all deficiencies be made up in the early years by general tax levy.

It is further recommended that the following prices be assessed per unit: For first assessment, 0.3c. per sq.ft., and 15c. per front ft.; for rental, 28c. per 1000 gal. entering the sewer; and that $8.40 be charged for unmetered connections, subject to a discount of 20% for sill-cocks.

The latter part of our comment on Mr. Snow's report, including a statement of some local conditions peculiar to Brockton, and a statement of two possible dangers connected with the adoption of the plan, was as follows:

Although this system has been worked out to meet the situation of Brockton, which is in a number of respects unique, the general principles involved may be applicable in other places. The special conditions at Brockton are as follows: (1) No sewers are yet in use, although the city has a population of about 30,000; this renders possible the adoption of any desirable system without the unfairness, real or fancied, which follows a change in the case of old systems. (2) It is expected that the whole city will be sewered in a comparatively few years, so that the total cost of the system can be readily estimated, which is essential to this plan. (3) The large percentage of metered water taps and the fact that the city owns the water-works, so that water and sewer departments can coöperate, while each desires to keep the water consumption down, are favorable to a yearly rental plan, based on water consumption. Modifications of these conditions might make the system difficult of application or might cause friction when applied. Nevertheless the principles at the bottom of it seem correct, and this general distribution of the burden of a sewerage system, whether in these or other proportions, seems fair and likely to be popular. General public and individual private benefit are each recognized and the latter is divided into two classes, (1) potential benefit through increased value of a certain piece of property because the sewer passes by it and may be used, and (2) the actual benefit through use.

Two dangers which beset the extremes of the two methods most usually employed to raise money for sewers seem likely to be counteracted to a large extent by this plan: (1) When the whole cost is put in the general tax levy or is met by bonds, the interest and principal of which must be met by taxes, it is difficult to secure money for extensions, every taxpayer wishing to keep the rate down, and those living on sewered streets having no direct interest in extensions. In the Brockton plan the tax levy is increased by only one-fourth the total cost of the sewers, the bulk of the expenditure being put upon those whose land is improved or those who, by use of the sewers, are saved the expense of cleaning cesspools or privy vaults. The taxpayer feels that the sewerage system is, to a large extent, self-sustaining, like a municipal water-works plant. (2) The other danger is that where property benefited bears the whole expense of sewers it will, in case of assessments for frontage or area, try to keep the sewer out of the street. But in Brockton a given property owner will be paying toward one-fourth the cost of the system whether the sewers are in his street or not, and once in his street he need pay only an additional one-fourth for property benefit, unless he wishes to connect with the sewer. The sewer once in, however, he will already be paying toward one-half of the total cost of the system and the additional expense for the use of the sewer will seem small. Moreover, by economy in the use of water his rental may be kept low, and most people do not consider themselves extravagant water users.

A third extreme which may be considered, although it rarely occurs in practice, except in the few instances where sewerage systems are owned by private companies, is the levying of the whole cost upon the user in the form of a yearly rental for discharging into the sewer. This plan, when adopted, is likely to result in a comparatively heavy expense to the users of the sewers, as all not connected with the system contribute nothing toward its cost and maintenance. It discourages sewer connections, especially from owners of small houses and of tenement property. In Brockton only 50% of the cost of sewerage construction and maintenance is levied on the users, and the sums so raised are based on the future as well as the present use of the sewers, which tends to lessen the individual payments, while the users know that all the property in the city is bearing a part of the burden.

Altogether the Brockton plan has much to commend it and its practical operation will be awaited with interest. Mr. Snow has given to this subject the same patient and exhaustive study that he gave to the designing of the sewerage system, especially the purification plant, and he deserves high commendation for both pieces of work.

It is interesting to note that the "Brockton plan," as the scheme proposed by Mr. Snow and subsequently adopted by Brockton was called, was upheld, first by the Superior Court of Massachusetts, and finally by the U. S. Supreme Court (see brief note on p. 33 of our issue of July 18, 1901). We cannot state whether the exact divisions of benefit and rates of charge advised by Mr. Snow were adopted, but the frontage and area assessment and the quarterly use-rental were put in force.

In the light of the foregoing statements let us see how an assessment for a large district-outlet sewer was made in an important northwestern city and how the assessment was modified by the court (presumably a local court).

The practice at Duluth, Minn., the city in question, we are informed by Bert Fesler, city attorney, is for the Board of Works to make sewer assessments, give hearings thereon, and then confirm the assessment. Subsequently property owners have the right to appeal. Mr. Fesler outlines a case in point as follows:

In this case the sewer was an outlet sewer and the board of public works spread its assessment over the entire drainage district on the theory that outlet benefits were equal over equal drainage areas. In other words, assessed each square foot of ground within the drainage district exactly the same amount for benefits. It happened that some of the property assessed was about three miles from the sewer. As the sewer itself ran through forty acres or so of unplatted garden tracts or farm lands, there was at one point an area of three miles in extent of so called "farm lands" that were assessed for these outlet benefits.

The court modified the assessment by reducing the more distant lands, all unplatted and claimed to be farm lands, $1.50, $2 and $2.75 per acre from the $5.47 per acre which was the flat assessment of the board of public works, thus reducing the total assesments about $1000 out of about $10,000 appealed from.

Both the board of public works and the court made their assesments strictly on the theory of benefits: the board on the theory that distance from the outlet sewer did not affect the question as long as the land assessed was within the drainage district, and the court on the theory that that rule should be modified somewhat by consideration of the element of distance as affecting the reasonable probability of use of the outlet

sewer by the property assessed. Under our practice the trial before the court was not in the nature of a review, strictly speaking, of the finding of the board, but was a trial by the Court de novo. The law under which both were working required that the assessments should not exceed the benefits, that they should not exceed the cost, and that the assessments should be in proportion to benefits.

It will be noticed that the points involved would not be applicable to paving assessments or direct-benefit sewer assessments.

Mr. Fesler does not specifically state whether the total cost of the sewer was assessed against the property, but the inference is that that was the case, and that the city was to bear none of the expense. As originally levied, the assessment made no allowance for differences in benefit, it having been assumed that the sewer would be equally useful to all the area in the district. The court reduced the assessment materially on the more distant land, but further than this it did not go. The basis of proportioning the reduction is not stated, but as it is in even quarter dollars it was presumably more or less arbitrary.

Under the modified plan, distance governs the assessment rate, and in so far as actual use is recognized as a factor it is measured solely on what might be termed a time-distance basis; that is, the more distant the land the longer before it will be built upon and therefore the less should be the assessment rate. But only three zones are established. A property owner within a given zone who builds a house and makes a sewer connection 5 or 10 or 20 years soon than do his neighbors pays no more for the use than they do for the non-use of the sewer.

Of course, the sewer-assessment problem is beset with difficulties, some of which are pointed out in the quotation from our editorial of 18 years ago. The greater the difficulties, however, the more important is it that they be met and solved in the light of the best judgment and experience the country affords. Instead, it is usually left to some board or commission which never gave the subject a moment's thought until by chance it comes before them for consideration and decision. Such criticism as is justified is not to be directed so much against the commissioners, in such a case, as to charter framers, who fail to provide for a technically informed body to decide highly technical questions. But there is no law against a sewer commission or board of public works manfully confessing its ignorance by referring the whole subject to a competent engineer for investigation and wisely adopting his findings, as was done at Brockton.

The International Navigation Congress

Not for many years has so large a body of eminent foreign engineers visited America as was assembled in Philadelphia last week to attend the 12th session of the International Navigation Congress. International congresses in the United States have been few, in fact, compared with their frequency in Europe. The 3000 miles across the Atlantic forms a difficult barrier of time and expense to the holding of any large and representative international meeting of engineers in the United States.

The previous notable engineering congresses in this country have been those in connection with the Chicago and St. Louis expositions and the gathering of iron and steel engineers from England and Germany in October, 1890.

This year America is to be host to no less than four International Congresses. Besides the one opened last week, there are to be Congresses on Testing Materials, on Hygiene and Demography and on Chemistry.

The Congress of Navigation is notable because of the prominent part taken in it by the United States government. The participants in the Congress are men engaged in or interested in matters in connection with waterway improvement and maintenance, rivers, canals, harbors and terminal works; and almost without exception in all countries of the world this work is done by the government.

A very large proportion of the members of the Congress, therefore, are government officers and are in fact in many cases officially delegated to attend the Congress. As the United States has participated in previous congresses and its delegates have enjoyed the hospitalities of other countries, it was necessary for our national credit that courteous entertainment should be provided this year for our foreign visitors. Congress, fortunately, appreciated this and provided an appropriation of $50,000 to entertain the visitors. The City of Philadelphia, which was chosen as a location for the Congress, added as much more and the State of Pennsylvania half as much more, so those in charge of the Congress have had sufficient funds to entertain our foreign guests in a fitting manner, if not with the lavish hospitality which generally characterizes the entertainment provided by European countries for international congresses meeting on their soil.

It should be said, too, that other cities visited by the guests during the numerous excursions which have been arranged, have raised their own funds to cover the expense incurred by local committees in transporting the visitors to points of interest in the city.

Not only did the federal government make provision of funds to entertain the visitors, but it assigned an official of the Corps of Engineers, Col. J. C. Sanford, to make all arrangements in connection with the Congress and the Congress was formally opened by President Taft himself.

The foreign delegates to the Congress began to arrive at New York on incoming foreign steamers on Friday, May 17, and for a week thereafter delegations were brought on nearly every arriving steamer. A special committee of the American Society of Mechanical Engineers, assisted by members of other societies, met the visitors at the pier as each steamer arrived and assisted them in getting their baggage through the customs and forwarded to destination, and in finding their hotels or starting them on the way to Philadelphia, where the local reception committee met them on their arrival.

The formal opening of the Congress took place on Thursday morning, May 23, in Philadelphia's spacious Metropolitan Opera House. The members of the Congress, foreign and American, were taken in automobiles from the hotel headquarters to the opera house in a long procession, with the automobile carrying President Taft and Rudolph Blankenburg, Mayor of Philadelphia, at the head.

Congressman J. Hampton Moore, of Philadelphia, President of the Local Committee, presided and introduced the several speakers, the first being Mayor Blankenburg, who delivered his address of welcome to the delegates, first in French, then in German and last in English.

Governor Tener of Pennsylvania, prevented by illness from being present, was represented by Attorney-General John C. Bell, who referred to Pennsylvania's four waterway problems, viz., the deepening of the Delaware Channel, the provision for public wharves on Lake Erie, the prevention of floods at Pittsburgh and the prevention of stream pollution.

A noteworthy paragraph in the Governor's address was the following:

> With respect to our inland rivers the state, by the organization of the Department of Health and the Water Supply Commission, has committed itself to the public policy of supervised utilization and conservation of these inland streams, and the preservation of their purity.
>
> I believe that we should enlarge the power, authority and duties of the Water Supply Commission, so as to provide for the supervision and regulation of the erection, maintenance, inspection and safety of all dams, reservoirs, embankments and obstructions of any kind heretofore or hereafter erected or maintained, in, across or along any of these rivers and streams.

President Taft in his address emphasized the value of such international conferences in the promotion of peace and alluded especially to the contribution to the world's welfare made by the United States in the construction of the Panama Canal. He also dwelt upon the fortunate

choice of a lock canal as the type to construct at Panama. Quotations follow:

It gives me great pleasure to attend any Congress that is for the promotion of the peaceful arts between the nations of the world. We have been trying to do something in the matter of bringing about universal peace in this country, and we had the assistance of foreign nations in that matter. For the time being we are suffering a little halt, but I wish to say to the representatives of foreign nations abroad that the hearts of the American people beat high for every step toward universal arbitration and peace and the abolition of any method of settling international controversies save by peace and by judicial decision.

Perhaps in the mere engineering problems of treating rivers and streams and lakes we do not suffer in comparison with other countries. But there are certain phases of the navigation problem that to us are most important and in respect to which I do not think I state it too strongly when I say we are very backward. As a nation we like to go fast. When the railway came in, the slower navigation of the river and the canal and the lake did not attract us so much, and you who go more deliberately, more thoroughly, possibly more scientificially, were able to unite the advantages of water transportation with railway transportation in such a way as to use both to the height of their usefulness. We are greatly lacking in that kind of terminals that are necessary in order to make the two operate economically, usefully and rapidly. We have, it is true, one waterway on the lakes that perhaps does a greater business than any waterway in the world, and what we have to learn is that the use of waterways must be more or less adapted to the circumstances that surround those waterways and must in the nature of things govern their use. That is that you must have at the end of the waterway something to be carried to the other end, and then at the other end something to be carried back to the first place. If you haven't either, and put a waterway there, it is not going to furnish its own traffic.

I could wish very earnestly that this city of Philadelphia were nearer to the great engineering work of navigation which this nation is engaged in at Panama.

We speak generally of the Panama Canal and hardly realize that when DeLesseps first planned a canal across that Isthmus it was to be but 72 ft. wide and 28 ft. deep. Now the canal at its narrowest point is 300 ft. across the bottom, and reaches out into the lake, and there has a channel of 1000 ft., more than two-thirds of its length, with a depth of 45 ft.

The canal has also settled a controversy that we assumed the responsibility for deciding in advance, and I think has settled it in favor of those who rendered the decision. That is, that a sea-level canal would have been impossible, at least practically impossible; that the conditions are practically prohibitory. The slides which have occurred in Culebra Cut, the enormous quantities of water that have discharged themselves through the Chagres River, all show that the time which would have been taken and the money which would have to be expended in order to complete the canal as a sea-level canal would have been so great that even the energy of the American people would have halted and their interest would have lagged in view of the length of time that the work must have dragged on.

The construction of this canal by the United States at its sole expense is an evidence of the willingness of the United States to do something for the welfare of the world. It is a substantial evidence of the desire of the United States to make closer our relations with all the world, and to show to those who are here now as foreign representatives that there is a moving element among the American people that are determined to accept every evidence of brotherhood in you toward us and to show you the same feeling on our part toward you until we shall have reached a point when we can discard all battleships and meet as brothers.

Following President Taft's address, Gen. Wm. H. Bixby, Chief of Engineers, U. S. A., on behalf of the American Section of the Congress, gave formal welcome to the visitors.

Reply to these addresses of welcome was made by Prof. V. E. de Timonoff, of Russia, Acting President of the Association, and we select from his remarks the following interesting passages:

It has often been stated that nationalism was the miracle of the last century. The unification of peoples along race lines into recognized nations has indeed been a marked characteristic of the nineteenth century. The twentieth century seems likely to be distinguished for internationalism.

Each international association brings nations closer together, discovers their qualities, creates personal friendship among their representatives and increases their mutual esteem. Every international gathering certainly drives several nails into the coffin of war.

The International Association of Navigation has the right to boast of being one of the most important in this respect. This Association has practically existed since 1885, when the first Congress on Navigation was held in Brussels, and it became a permanent institution in 1898.

Congresses of Navigation have been held since in Belgium, Austria, England, France, Holland, Germany, Italy and Russia. The Association has about 2000 permanent members. It is encouraged by 47 countries, the yearly income being more than 100,000 francs.

No other great work now being carried on throughout the world is of such far-reaching and lasting importance as the Panama Canal. Never before has a work of this kind on so great a scale been attempted. Never has any work of the kind, of anything approaching the size, been done with such efficiency, with such serious devotion to the well-being of the innumerable workmen, and with a purpose so lofty and so practical. No two men in the service of any government represent a higher, more disinterested, and more efficient type than the two men at the head of this work—Colonel Goethals, the man who is actually doing the digging, and Dr. Gorgas, who has turned one of the festering pest-holes of the world into what is almost a health resort.

To have an idea of the importance and difficulty of what is being done now on the Isthmus, it is sufficient to compare the natural excavation of the cañon of Niagara to that of the artificial cut of Culebra. Both excavations are similar in size, but Nature required not less than 35,000 years to make the Niagara cut, and the Americans will make the Culebra cut in six years.

At the conclusion of the exercises the foreign delegates were presented to President Taft, and the Congress was then entertained at luncheon by the City of Philadelphia.

Some of the technical papers presented to the Congress are printed elsewhere in this issue. A further account of the proceedings we postpone till next week.

The Panama Canal Terminals on the Atlantic side include plans for two dock or quay walls on Cristobal Point, and a number of piers jutting out into the ship basin from a mole now in the process of construction. It is intended to build at present only two docks, a part of the mole and one pier, leaving the remainder of the work to be executed as the need for it develops. All the dock walls will be built of reinforced concrete on caisson foundations, and work on the two mentioned has already been begun. These are Docks 15 and 16. The wall of Dock 15 runs along the south side of Cristobal Point, and the wall of Dock 16 on the west side. Pier 17 juts out from the mole in a direction parallel to the wall of Dock 16, giving a least width between of 300 ft. The wall of Dock 15 is 500 ft. long; that of Dock 16 and the length of the pier is 1100 ft. The pier is to be 209 ft. wide.

The committee in charge of the canal terminals recommends that each pier be provided with four railway tracks running the full length of the pier, two depressed tracks in the center and two apron tracks at the floor level. Pier-sheds of reinforced-concrete or concrete-incased steel the entire width of the pier to within not less than 18 ft. of each face of the pier are recommended. Pier-sheds are to be one story and have not less than 25 ft. of clear height.

In regard to furnishing the piers and quay walls with cranes and similar cargo-handling devices, since the amount of freight which will break bulk at either terminus of the canal for transshipment by rail across the isthmus will be very little, costly mechanical freight-handling appliances are to be avoided if possible. Although space will be left between the sheds and the pier for such installations if they become economically necessary, the committee recommends that for the present all freight be handled by the vessels' cargo booms; single pieces too heavy to be thus handled can be moved by floating crane boats.

At Balboa, on the Pacific side, the situation is somewhat different on account of the greater range of tides and the difference in the type of vessels entering the port at the present time. The committee believes that this difference will be compensated for by the six new electric cranes under contract for the old French wharf at Balboa, supplemented by such of the present cranes as are serviceable.

The transfer of freight on piers will be by hand or electric trucks, depending upon the distance and other conditions; transfer between piers will be by standard-gage cars or by electric trucks.

Notes on the Digestion of Sewage Sludge

By Charles Saville*

The recent completion of the sewage-disposal works at Baltimore, Md., is a matter of considerable interest to sanitary engineers and municipal authorities, as these works, the preliminary designs for which were made in 1906, are the only ones of any magnitude in this country where separate tanks are provided for digestion of the sewage sludge.

There are, as is now well appreciated, many advantages in providing for the rapid and complete digestion of sewage sludge. In this connection "digestion" refers to the thorough decomposition, or rotting out, of non-resistant organic matters which are easily putrescible and thereby capable of creating a nuisance.

The quantity of sludge deposited from sewage is large. In a fresh condition, and containing 90% or more of water it sometimes amounts to as much as 750 cu.ft. per month per thousand persons connected with the sewers, this being the figure reported from Elberfeldt, Germany, where the sewers are built on the separate system. Furthermore, fresh sludge usually has an objectionable odor which may continue to be noticeable for a long period. Because of these two characteristics, the disposal of fresh sewage sludge, particularly in the vicinity of thickly settled communities, is a difficult problem.

With well digested sludge on the other hand, the volume to be handled is comparatively small, being, under certain conditions, less than one-fifth as great as with fresh sludge. This great difference is due to the smaller water content of the digested sludge and to the breaking down or rotting out of the easily putrescible organic matter, part of which passes off in the form of gases. The extent to which fresh sludge is reduced in volume by digestion depends on numerous factors, including the nature of the organic matter in the sludge, the amount of mineral matter present, the condition under which the digestion takes place, the length of the digestion period, etc., etc.

It is also true that well digested sludge has no objectionable odor, because the easily putrescible organic matter in it has been thoroughly rotted out. Under certain conditions digested sludge flows easily. Furthermore, on account of the entrained gases of decomposition it dries quickly to a porous earthy mass which can be readily handled and easily disposed of.

It is thus evident that there is much to be gained from a complete digestion of sewage sludge, and throughout the past 40 years considerable time and thought has been spent in attempts to find the most efficient means of bringing about the desired results.

Perhaps the best known method of effecting the digestion of sewage sludge is the septic tank, in which the deposited sludge is allowed to digest in contact with the flowing sewage. Under these conditions the rate of digestion is usually not high, and as the digestion of the sludge makes the sewage itself septic and malodorous, and at the same time interferes with the settling action, the ordinary septic tank is gradually going out of favor.

The next step in advance was taken at the State Experiment Station, in Lawrence, Mass., by removing the deposited sludge from the settling tank to a separate digestion tank. This method of sludge treatment was first tried some 12 or 13 years ago, and has since been adopted on a practical scale in England as well as at a few places on the Continent; but only to a small extent in this country. In some instances it has proved comparatively successful, but at a number of places the results have not been entirely satisfactory; in Germany, for example, as reported by Prof. Thumm, in his recently published book on sewage disposal.* Similar unsatisfactory results have also been reported from the sewage-testing stations at Philadelphia and Chicago. It is not strange, therefore, that recent years have witnessed the search for a still better means of bringing about the desired digestion.

This has gradually resulted in developing, first in England and then in Germany, a tank with two chambers; one, through which the sewage flows and in which the suspended matters settle out; the other, for the storage and digestion of the sludge, being so arranged that not only the sludge, but also the products of digestion, cannot have an injurious effect on the sewage in the settling chamber. In this type of tank, in the form in which it has been developed in Germany, there is no flow of sewage into or through the sludge chamber. Consequently the organic matter in the sewage standing in the sludge chamber soon becomes decomposed, after which the digestion of the sludge takes place in a liquid which is practically free from easily putrescible organic matter. Such tanks are spoken of as the Emscher or Imhoff tank. They have been operating on a practical scale in Germany for six years and have given uniformly good results. In this country there are as yet only a few in operation; but wherever such tanks have been given a thorough test on a small scale with ordinary city sewage, the digestion of the sludge has been excellent, and a large number of municipalities throughout the country are now planning to use them.

The time thus seems appropriate for inquiring into the relative advantages of the separate sludge-digestion tanks and the Imhoff tanks.

In discussing this question the writer would suggest the advisability of keeping in mind the following points of view:

(1) Can separate sludge-digestion tanks be made to give as good results as Imhoff tanks, with respect not only to the character and quantity of the digested sludge, but also to the effect on the settled sewage and subsequent processes of purification, and to the production of objectionable odors, both during the settling of the sewage, and during the digestion and subsequent disposal of the sludge?

(2) Will the digestion of the sludge be as rapid in separate sludge-digestion tanks as in Imhoff tanks, and will the cost of construction and operation be as low?

Sludge which has been thoroughly digested in separate tanks probably does not differ materially as to character or quantity from that which has undergone digestion in ordinary septic tanks or in Imhoff tanks; but with the type of separate digestion tank thus far used the removal of the sludge from the tanks has not proved as easy as with Imhoff tanks, and in some instances it has been found difficult (with separate digestion tanks) to bring about a satisfactory digestion of the sludge.

Small-scale tests with a separate sludge-digestion tank at the Philadelphia Sewage Testing Station, gave disappointing results, as may be seen by reference to page 171 of the report on the tests, where the opinion is expressed that "the placing of sludge in open water-tight tanks is not an advantageous means of disposal." In these tests fresh sludge was added to the digestion tank every two weeks, and although there was some gas formation and liquification during the first part of the test, this ceased altogether with cold weather, which may be explained in part by the fact that the tank was quite shallow. It is said that the tank always had a thick scum and that there was no material change in the character of the underlying sludge.

The excess liquid which overflows from separate digestion tanks will be of a peculiarly objectionable character and must be purified. If discharged back into the settling tank it would tend to make the sewage septic, and in any case presents a difficult problem for solution.

The sludge, as discharged into separate digestion tanks, will always contain some sewage having undecomposed organic matter in solution, so that the process of digestion is likely to be some-

*Of Hering & Gregory, Consulting Engineers and Sanitary Experts, 170 Broadway, New York.

*Handbuch der Hygiene, II Band, 2. Abteilung, Wasser und Abwasser. Reviewed on p. 38, of Engineering News, of Nov. 16, 1911.

what similar to that occurring in ordinary septic tanks, with consequent probability of the production of objectionable odors. These odors, as pointed out by Langdon Pearse, of the Sanitary District of Chicago, are likely to be especially noticeable at times when the sludge is being discharged into the digestion tanks, for the reason that the sludge, having already begun to decompose, contains foul-smelling gases, which, due to the stirring up of the sludge, will escape. The separate sludge-digestion tank at the Chicago Testing Station, is said to have produced a very noticeable putrid smell, and engineers who have inspected, during warm weather, the operation of the separate sludge-digestion tank at Forest Park, Baltimore, report somewhat similar conditions.

Another point of importance in this connection is that such separate digestion tanks as have been built have a larger surface of liquid exposed to the atmosphere than is the case with Imhoff tanks, and may thereby give a greater opportunity for objectionable odors to become noticeable.

In Birmingham, England, where sludge from septic tanks was deposited to a depth of 5 or 6 ft. in large open lagoons, Mr. Watson stated that a thick, strong crust was formed on the surface, beneath which the sludge remained in its original condition, with no apparent loss in amount, and continued to give off foul odors irrespective of the age or density of the deposits. It would, no doubt, be possible to mention other instances in which separate tanks have not been successful in bringing about a satisfactory digestion of the sludge. It should be stated here, however, that experiments now being made in Birmingham under Mr. Watson's direction, indicate that separate sludge-digestion tanks, when operating with Birmingham sludge under certain conditions, can be made to give a well digested sludge.

The effect of this method of sludge treatment on the flowing sewage is likewise a matter of considerable importance. If the sludge is removed from the settling tank under water, this must be done at frequent intervals (say a week or sometimes even less, in warm wather), in ordr to prevent a partial decomposition of the organic matter in the sludge previous to its removal from the settling tank. Such partial decomposition in the settling tank, by stirring up the deposits, interferes with the settling action and has the effect of making the sewage septic. This may lead to the production of objectionable odors around the settling tank. Unless the sewage, as well as the sludge, is removed at intervals from the settling tank and the entire tank thoroughly cleaned, there is pretty sure to be trouble of the above mentioned variety, as it is difficult even with specially designed tanks, to remove from the settling tank under water all of the sludge.

The second point of importance to keep in mind is whether the digestion of the sludge can be effceted as quickly in separate tanks as in Imhoff tanks. If the digestion is slower the tanks must be larger, which, with this method of treatment, is an important factor to be considered, as it is necessary to have two distinct sets of tanks.

The digestion of sewage sludge is a biological process and without doubt proceeds rapidly only when the environment has been made suitable for the particular kind of bacteria which do the work, and when other unnecessary types of bacteria are so far as possible excluded. The creation of the proper environment requires time, and after the environment has once been created it is desirable, just as with a sprinkling filter, to have the food supply, in this instance consisting of sludge particles, enter the tank in a continuous stream and at a uniform rate. This is not easy to arrange with separate sludge digestion tanks. Consequently every time a new supply of sludge is added to the tank "the balance" of the bacterial activities is upset, partly because the newly added sludge, including some sewage and numerous varieties of bacteria, constitutes such a large proportion of the total contents of the tank.

In many of the separate digestion tanks, thus far operated, no provision has been made for having, above the sludge deposits, a considerable depth and volume of water in which the sludge can be kept, by the rising gases, in a more or less continuous state of motion. The bacterial toxins can, therefore, not escape easily nor are fresh sludge surfaces exposed to bacterial action as frequently as desirable, all of which tends to check the rapidity of digestion.

To sum up, it seems probable that although separate sludge-digestion tanks *can* be made to give sludge of a satisfactory quality, the time required for complete digestion is nevertheless likely to be longer than with tanks of the Imhoff type, thus calling for larger tanks. There is also a greater possibility of the production of objectionable odors.

The cost of operating separate sludge-digestion tanks is, in most instance, likely to be higher than with the type of tank which includes both a settling chamber and a sludge-digestion chamber. In the first place, as already pointed out, the best results with respect to the effect on the sewage itself can be obtained only when the settling tank is completely emptied and cleaned at regular intervals. Secondly, it is questionable whether fresh sludge can be removed as easily under water as completely digested sludge. Furthermore, the removal of sludge from one tank to another always involves loss of head, and in many instances, may necessitate pumping. Finally, the removal of sludge from some separate sludge-digestion tanks, even after more or less complete digestion, has been found to be a slow and expensive process. At the small disposal works in Forest Park, Baltimore, for example, the writer saw four men at work shoveling the digested sludge out of the tank. In cases where this method of removal has to be adopted, the construction of an extra tank may be necessary, since one tank will be out of commission much of the time. The necessity for purifying the liquid which overflows from the separate digestion tanks has already been mentioned.

Although separate sludge-digestion tanks have certain disadvantages, their adoption, under some conditions, may prove desirable. In such cases, it would be advantageous to make the tanks as deep as possible, so that the digested sludge when removed will have a comparatively low water content and contain a part of the gases of decomposition. When first placed in operation, it would be desirable to fill the tanks with clean water, containing no easily decomposable organic matter. This will tend to lessen the production of odors, particularly during the "ripening time." The fresh sludge should be discharged into the tank at regular intervals, and in comparatively small quantities, so that the amount of sewage accompanying the sludge will be small in comparison with the whole body of liquid in the tank, and will, therefore, not tend to make this liquid odorous.

The tanks ought never to be completely emptied, as it will then be necessary for them to pass once more through the ripening time. If the gas developed does not prove sufficient to keep the sludge well stirred up so that new surfaces are constantly being exposed to the action of the bacteria, it may be possible to add certain cellulose substances, which will increase the gas formation, or to stir the sludge artificially, with a paddle, with compressed air, or with clean water. In case water is used for this purpose, there would be an overflow from the tank, which would probably require purification.

It is hoped that those who have had experience with, or knowledge of, the operation of separate sludge-digestion tanks, will contribute to a discussion of their relative advantages or disadvantages, so that, before forming definite opinions, engineers may have the benefit of as many as possible of the available data.

An International Rubber Congress and Exhibition will be held at Batavia, Netherlands East Indies, in April, 1914. Attention will be given to all matters concerning rubber production and the preparation of the crude product, together with other related industries such as the production and preparation of balata, jelotong and gutta-percha. This will be the first international rubber exhibition to be held in one of the important rubber-producing countries.

New York City Subway Planning

The badly needed extension of rapid-transit facilities is New York City has been the subject of a remarkable series of negotiations during the two years just past. The negotiations now appear to be drawing to a close, by substantial agreement between the municipal authorities and the two private companies concerned, upon the construction of a large network of new subway and elevated lines, aggregating 105½ miles of road, or 309 miles of single track, about half of which is in subways. While we have printed occasional notes on the progress of the discussion between city and companies or among the authorities, most of the discussion has been too aimless, or too partisan and unpromising, to warrant printing for the information of readers outside of New York. The present stage of planning, however, is definite and promising. The city's administrative body, the Board of Estimate, has agreed with the Public Service Commission and with the two companies concerned, both on a location plan of new lines and on the principal terms of the contracts and franchises for construction and operation.

The first essential point of the discussion and negotiations referred to was the question whether the city should construct and operate future rapid-transit lines (subways in settled districts, elevated railways in outlying territory) on its own account, or whether it should preferably find a private company to construct and operate them. The discussion of this point was very acrimonious and partisan: there was a strong interest in favor of city construction (and to a lesser extent also in favor of city operation), while a very active element, in which the mayor, W. J. Gaynor, forms a chief figure, stood for construction by private companies and operation by private companies. The exertions of this latter party were strongly aided by the fact that the city's legal borrowing capacity over and above its present bonded indebtedness was quite limited at the beginning of the negotiations, and, though in the meantime it has increased to some $150,000,000,* it is even now adequate for only part of the very extended construction program that is proposed.

A second point, no less important, was the radical difference of opinion between what we may call a competition party and a monopoly party on the question of private construction and operation. One party held to the belief that private operating contracts should be made only upon competitive bidding, the most favorable terms for the city to be accepted, irrespective of the bidder. The other party held that at almost any cost the new lines should be constructed and operated by the company which operates the present subway (Interborough Rapid Transit Co.). This latter party argued that the city's transit network should be developed as a unit, so that the citizens would in future have the benefit of universal transfers and a single fare. However, the Interborough company took the attitude of dictating terms under which it would engage in a contract for new construction, and objected to all of a series of propositions put forward tentatively by the city authorities, whereas the Brooklyn Rapid Transit Co. (and at one stage of the proceedings still another company) assumed an attitude indicating more of a desire to meet the city's terms, so far as these could be reconciled with the development of a series of lines commercially suitable for operating as a rapid-transit system.

The first stage in this discussion was closed about a year ago (June, 1911), when a specific proposition made by the city to both the Interborough and the Brooklyn companies was accepted by the Brooklyn company and rejected by the Interborough. Thereupon the Board of Estimate passed a resolution notifying the Public Service Commission that contracts with the Brooklyn company along the lines of the stated proposition would be accepted by that Board when presented to it. Shortly after this, however, the Interborough company proposed to the city to reopen negotiations. This led to very protracted discussions between representatives of the company and a special committee of the Board of Estimate (which since January, 1911, has had this matter in charge). The ultimate result was that in February, 1912, the Interborough company submitted an amended proposition involving better terms for the city, chiefly a somewhat lower share of the net receipts for the company before equal division of profits, and an abandonment of the company's previous demand for perpetual franchises for the proposed extensions of its elevated-railway lines.

After this proposition was received and considered by the Public Service Commission, the subject was further studied and the Board of Estimate's committee devoted its attention to laying out the entire system of proposed new lines in such a way as to give separate, self contained, partly competitive but still essentially supplementary, systems to the Brooklyn company and the Interborough company. The result of these studies has just been announced in a full report of the committee to the Board of Estimate, and the Board has adopted the committee's recommendations.

Proposed New Rapid-Transit Lines

Interborough System—This includes three essential elements:

(1) A Lower Westside subway line (*A* in the map herewith), running south from the present subway at 42d St. and Broadway through Seventh Ave., Varick St., West Broadway and Greenwich St. to a terminus at the Battery, with a Brooklyn branch *B* diverging from the main line at West Broadway and Park Place, running easterly through Park Place, Mail St., Spruce or Beekman St., William St., Old Slip, and by tunnel under the East River to Clark St. in Brooklyn, and along Clark and Fulton Sts. to a connection with the present Interborough Subway at Borough Hall. This line, if operated with the present subway from 42d St. and Broadway northward, will furnish a complete west-side subway line, with its own tunnel to Brooklyn, and with an extension into the heart of Bronx Borough by way of the Bronx branch of the present subway.

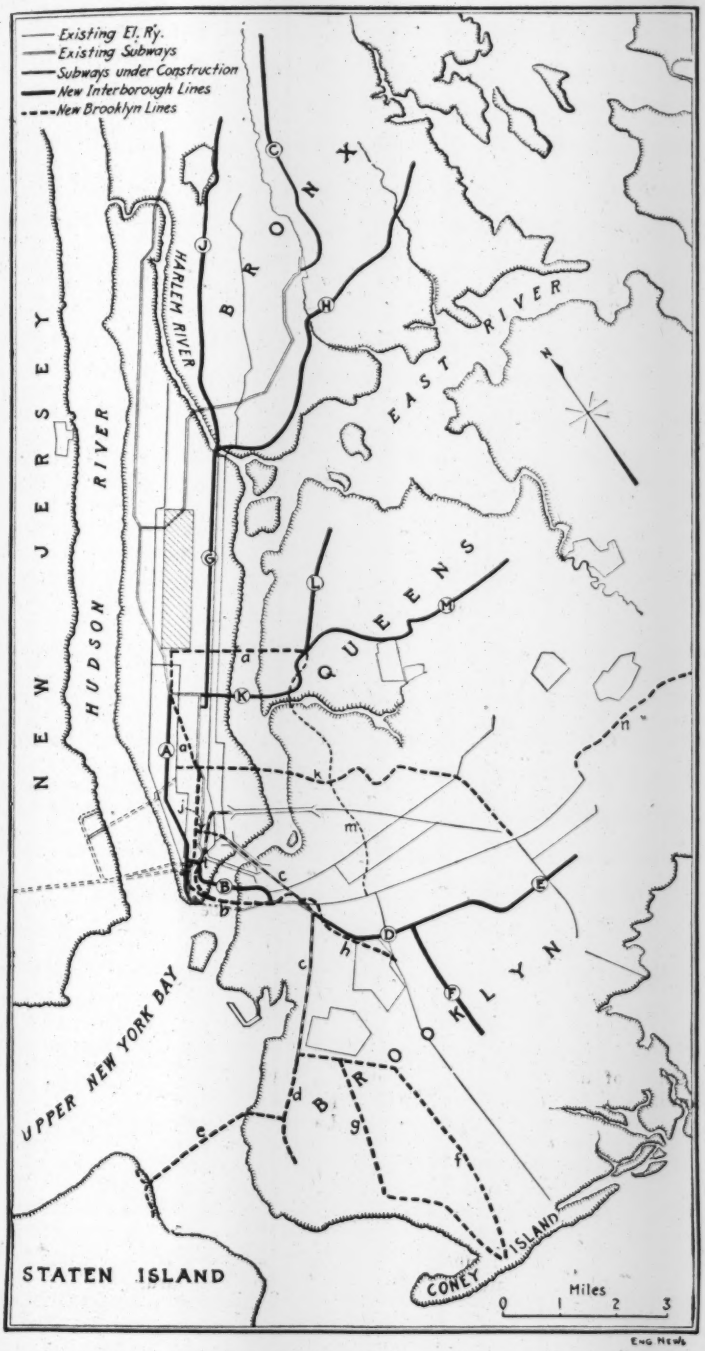
In connection therewith, the Bronx end of the present subway is to be expanded by a northward branch *C* from 179th St. north through White Plains Ave. to the city line. At the southeast, the Brooklyn end of the present subway is to be greatly extended, by a subway *D* along Flatbush Ave. from Atlantic Ave. southeasterly to Prospect Park Circle and thence easterly under Eastern Parkway, with an easterly branch *E* along east 98th St. and Livonia Ave. to New Lots Road, and a southerly branch *F* from Eastern Parkway through Nostrand Ave. to Flatbush Ave.

(2) An East-side subway line, *G* in our map, departing from the present Interborough Subway at Park Ave. at some point between 32d and 42d Sts., easterly to Lexington Ave., north under Lexington Ave. to the Harlem River, across the Harlem River by tunnel to 135th St. in Bronx Borough, and via Park Ave. and Mott Ave. to a connection with the present subway at 149th St.* This line, operated in conjunction with the present subway along Park Ave. and Lafayette St. southward to the present Battery Tunnel to Brooklyn, will give a complete east-side line for the Interborough company. Its northerly ends in the Bronx are to be extended several miles northeast and north by two branches: one, *H*, through 135th St., Southern Boulevard, Whitlock Ave. and Westchester Ave. to Pelham Bay Park; the other, *J*, northerly by way of River Ave. to 162d St., with an elevated extension along River and Jerome Aves. to Woodlawn Rd.

Thus, the most northerly portions of Bronx Borough at three different points

*In large part through a recent law exempting self-supporting bonds from inclusion in computing the difference between debt and borrowing capacity.

*The Lexington Ave. line is the north half of the much discussed "Triborough Subway," laid out by the Public Service Commission two or three years ago. It was composed of our line G, extended south to about Ninth St., and the lower part of line a, with connection j to the Brooklyn Fourth Ave. subway. Contracts have been let for most of the Lexington Ave. portion above 42d St., for the Harlem River crossing, and for parts of the lower Broadway section. Work is under way.



MAP OF PROPOSED NEW RAPID-TRANSIT NETWORK; AS JUST APPROVED BY BOARD OF ESTIMATE AND PUBLIC SERVICE COMMISSION

PRINCIPAL NEW LINES

Interborough Rapid Transit Co.

A Seventh Ave. subway;
B Old Slip Tunnel;
C White Plains Rd. elevated extension;
D Eastern Parkway subway;
E Livonia Ave. branch;
F Nostrand Ave. branch;
G Lexington Ave. subway (under construction);
H Pelham Bay branch;
J Jerome Ave. branch;
K Steinway Tunnel;
L Astoria branch;
M Corona branch;

Brooklyn Rapid Transit Co.

a Brooklyn subway;
b Whitehall St. Tunnel;
c Fourth Ave. subway (under construction);
d Fourth Ave. extension;
e Narrows Tunnel;
f g Coney Island branches;
h Prospect Park extension;
j Canal St. connecting subway;
k Fourteenth St. Tunnel to East New York;
l Centre St. subway extension;
m Elevated railway from Brooklyn to Queensboro Bridge Plaza;
n Jamaica elevated extension.

are brought into connection with the eastern and central portions of Brooklyn over two separate lines of the Interborough company, passing down the east and west sides of Manhattan.

(3) A line *K* from the central portion of Manhattan Borough across the East River to the Borough of Queens, via the (already constructed but as yet unused) Steinway or Belmont Tunnel, which is to be extended westerly by subway to 42d St. and Broadway, and is to be continued easterly to Queensboro Bridge Plaza. The easterly part is to have two branches: one, *L*, northerly through Debevoise Ave. to Astoria; the other, *M*, eastward through Woodside Ave. to Corona.

These lines are to be constructed jointly by city and company. In addition there are to be

(4) Several improvements in the elevated-railway system; and extensions of the existing elevated lines beyond their present terminals, to be constructed by the company alone. They include: Third-tracking (or completing the third-tracking of) the Second Ave. and Third Ave. elevated lines from City Hall to 129th St.; Four-tracking the joined line from here north to 149th St. in the Bronx, and (possibly) third-tracking the Bronx elevated line from here north to Bronx Park; Completing the third-tracking of the Ninth Ave. line between Rector St. and 155th St.; Extending the Ninth Ave. elevated line from its terminus at 157th St. and Eighth Ave. across the Putnam R.R. bridge over the Harlem River and by elevated railway and tunnel to a connection with the new elevated line *J* in Jerome Ave.; Constructing an elevated-railway connection between the Third Ave. elevated railway at 142d St. and the elevated portion of the existing subway near Park Ave.; Extending the Third Ave. elevated railway in the Bronx from Fordham Ave. through Webster Ave., Gun Hill Rd. and White Plains Ave. to a connection with the proposed northerly extension *C* of the existing subway; and finally, connecting the Second Ave. elevated railway at 59th St. with the Queensboro Bridge tracks for a connection with the new lines centering at Queensboro Bridge Plaza.

BROOKLYN COMPANY'S SYSTEM — The important feature of the system which the Brooklyn Rapid Transit Co. will have under the proposed plans is that it will enter Manhattan, for the first time. Not only this, but it will have two complete and independent loop lines through Manhattan, and both these will be double or figure-of-eight loops, crossing the East River by way of Queensboro (Blackwell's Island) Bridge, a new tunnel at 14th St., Manhattan Bridge, Williamsburg Bridge, Brooklyn Bridge, and a new tunnel at the Battery. The inner loop will use the Centre St. subway (now nearly finished) which connects the Manhattan end of the Williamsburgh, Manhattan

and Brooklyn Bridges and has now a terminal station under the new Municipal Bldg. The outer loop will utilize a new subway line which is to run under Broadway from the financial district in lower Manhattan through the business and theater districts, and, circling past Central Park along 59th St., will lead over the Queensboro Bridge, at whose easterly end it will make connections with Queens Borough in various directions, while at the southerly end of the Broadway line a tunnel at the Battery will make connection with all the transit lines of the central portion of Brooklyn.

The lines destined for the Brooklyn Rapid Transit Co. are:

(1) The Broadway subway, marked *a* in our map, beginning at the Battery at Whitehall St. and extending northerly through Whitehall and Morris Sts., Trinity Place, Church and Vesey Sts., Broadway to 42d St., Seventh Ave. to 59th St., and east through 59th St. to the Queensboro Bridge.

(2) The new Battery tunnel, *b*, at the south end of the Broadway line, will lead from Whitehall St. under the East River to Montague St. (Brooklyn), Fulton St. and Willoughby St. to a connection with the Fourth Ave. subway in Flatbusn Ave.

The Brooklyn Fourth Ave. subway *c* (now under construction, extending from the eastern end of the Manhattan Bridge down Fourth Ave. to 43d St.) is to be extended (*d*) to 86th St., with provision for branching at 65th St. to Staten Island, *e*, and eastward to two lines *g* and *f* leading to Coney Island. (A tunnel *e* under the Narrows to Staten Island is set down as part of the present project, but its realization is so remote that it need not be considered for the present, though we have indicated it on the map.)

This main loop line is to be supplemented by two branches: one, *h*, in Brooklyn, leaving the Fourth Ave. subway at Fulton St. and passing through St. Felix St. and under the Long Island R.R. station to Flatbush Ave. near Fifth Ave., thence to Prospect Park Circle and to a point of connection with the Brighton Beach line of the company's elevated-railway system at Malbone St.; the other, *j*, in Manhattan, connecting the Broadway route at Canal St. with the tracks of the Manhattan Bridge line, which in turn connects with the Fourth Ave. subway on the Brooklyn side.

A central stem *k* to this outer loop is to be provided by a crosstown line under 14th St., Manhattan, passing by a tunnel under the East River to North 7th St., Brooklyn, and through Metropolitan Ave., Bushwick Ave., Johnson Ave., and Wyckoff Ave., to a connection with the Broadway (Brooklyn) elevated line in East New York. This stem will in Manhattan cross the main Broadway subway, giving opportunity for either passenger transfer or track connection.

The Broadway loop line, therefore, has not only an outlet at its northerly and southerly ends but two intermediate stems, the important 14th St. stem and the Canal St.-Manhattan Bridge branch.

The inner loop is to be composed of the Centre St. loop subway (now building) and

(3) Extension of the Centre St. loop subway southward through Nassau St. and Broad St., *l*, to a connection with the new Battery Tunnel at Whitehall St.

The Centre St. line has track connection with the Brooklyn, Williamsburgh and Manhattan Bridges, and will have transfer or track connections with the Canal St. line and thus indirectly with the Broadway subway.

The Fourth Ave. subway in Brooklyn, which forms an element in both loop lines, is to be extended for suburban and seashore service by

(4) A combination subway and elevated route *g*, through 38th St., 10th Ave., New Utrecht Ave., 86th St. and Stillwell Ave. to Surf Ave. (Coney Island), A branch *f* of this same line will be developed over another route to Coney Island, utilizing an existing surface route.

(5) A number of elevated-railway extensions and new lines are to be undertaken by the Brooklyn Rapid Transit Co. at its own cost as part of the general project. They include: An extension, *m*, of the Brighton Beach elevated railway from Fulton St. and Franklin Ave. northerly through Greenpoint and Long Island City to the Queensboro Bridge Plaza; A line *n* to Jamaica from the terminus of the present Cypress Hills elevated, via Jamaica Plank Road; Extension of the Liberty Ave. elevated railway to Richmond Hill; and Construction of third track on the Fulton St., Broadway and Myrtle Ave. elevated structures. Several of the surface lines of the Brooklyn Rapid Transit Co. are also to be rebuilt and fitted with an increased number of tracks and so reconstructed as to remove various grade crossings. Further, a general elevated-railway terminal is to be built at Coney Island in place of the scattered terminals at present, and a connection point is to be provided at East New York.

Mileage of Rapid-transit System

The magnitude of the projected rapid-transit system, in relation to the existing system (but excluding the elevated-railway mileage of the Manhattan Ry. network, owned by the Interborough) is set forth by the table of mileage, below.

EXISTING AND PROJECTED RAPID TRANSIT MILEAGE, NEW YORK CITY.

Interborough system	Subway, miles of single-track	Elevated, miles of single-track	Total, miles of road
Present lines	56.1	16.9	25.9
New lines	75.6	71.5	48.5
	131.7	88.4	74.4
Brooklyn system			
Present lines	....	104.9	41.9
New lines built by company	1.6	84.1	30.2
New lines built by city and company	69.9	6.6	26.9
	71.5	195.0	98.1
Combined system			
Present line	56.1	121.8	66.9
New line	147.1	161.6	105.6
Total	203.2	283.4	172.5

Note—These figures do not include the Manhattan Ry. Co.'s elevated lines, which aggregate 37.7 miles of road, or 115 miles of single-track. If these are added the new system will bring the rapid-transit road mileage up to 210.2 (against 104.6 present), and the single-track mileage up to some 650 miles (against 295.9 miles at present).

When this table is considered in connection with the following table of costs, the fact should be noted that the cost estimates are in part approximate, and will probably be overrun considerably. But on the other hand some items included, as the Staten Island tunnel, will probably not get beyond the project stage for many years to come.

Terms of Construction and Operating Contracts

Construction—All the new subway lines are to be built by the city and the respective company jointly, the companies obligating themselves to furnish specified sums while the city furnishes the remainder. Taking into account the extensions constructed by company money, the following table summarizes the money division of construction:

ESTIMATED COST OF NEW RAPID-TRANSIT LINES

Interborough System		
City money	$59,649,250	
Company money	56,000,000	
Total Interborough cost		$115,649,250
Brooklyn System		
City money	64,722,950	
City money already spent on Fourth Ave. and Centre St. lines	28,486,000	
Company money for new lines	13,000,000	
Company money for extensions and improvements	21,000,000	
Total Brooklyn cost		127,208,950
Combined system, construction only		$242,858,200

SUMMARY

1. City money for new lines	$124,372,200	
2. City money for lines partly built	28,486,000	
3. Company money for construction work	90,000,000	
Total construction		$242,858,200
4. Company money for equipment Interborough	21,000,000	
B. R. T.	26,000,000	
Total equipment		47,000,000
Grand total, construction and equipment (including Fourth Ave. and Centre St. loop lines now being built		$289,858,200

All the construction is to be directed by the Public Service Commission and all contracts for joint work of city and company are to be let by the commission and under its plans. The money to be contributed by the companies for the cost of

these lines is to be paid over to the city authorities and expended by them. The contracts for the elevated extensions and other work to be done by the companies at their own expense will, of course, be let by the companies, but the plans for such work must be approved by the Public Service Commission. Incidentally, on account of the need for certain valuations for the city purchase clauses to be introduced in the contracts, the Public Service Commission is to make a complete investigation of the elevated-railway structure and other properties of the Manhattan Railway Co. (owned by the Interborough Co.).

It is understood that the Interborough Co. will be financed by J. P. Morgan & Co. and associates, while the Brooklyn Rapid Transit Co. will be backed by Kuhn, Loeb & Co.

OPERATING TERMS—The terms agreed upon as a basis for the operating contracts are quite complex and numerous, but certain of their most important elements may be stated as follows:

For each company's system the rate of fare is to be 5c., and universal transfers are to be granted on each system for itself.

The lines owned by city and company jointly are to be amortized at such a rate as to extinguish the capital charges before the expiration of 49 years, and the lines are to become the sole property of the city at the end of 49 years without further payment. The city may, however, buy any part of these lines after the expiration of 10 years from beginning of operation, the price being fixed at construction cost (plus 15%) less the amortized proportion.

The net receipts of operation (after deducting from gross receipts the operating expenses, provision for depreciation and renewal, taxes, insurance, general and administration expenses, etc., are to be utilized in the following sequence:

(1) In the case of the Interborough only, the city receives an amount equivalent to the rental payable under the existing subway contracts; (2) the operating company receives a sum representing the average net earnings from the operation of the existing line, and out of this amount the company must pay interest, sinking fund, and other fixed charges on its present investment; (3) the operating company receives a yield of 6% on its new investments; (4) the city receives an amount equal to its bond-interest charges, plus 1% for sinking fund; (5) the remainder is divided equally between city and company. However, an important provision is that any deficiency in any year in meeting the payments to the operating company shall be paid off in subsequent years before any payments are made to the city. Similarly any deficit in the interest and sinking-fund payments to the city must be carried along and paid off in subsequent years before the remaining profit is divided equally.

A further important feature of the agreement is that the leases for the existing subway are to become coterminous with the leases for the new lines, that is, they will end 49 years after commencement of operation of the new system. Further, if the city buys the new lines at any time after 10 years as provided, then it may after 35 years acquire the existing subways upon payment of a sum equal to the remaining value of the lease.

The elevated-railway extensions are to be held by the companies under leases or franchises of maximum duration 85 years. The city has the right to acquire these lines after 10 years or at the termination of the franchise, but it is specified, curiously enough, that the city must not use the lines for rapid-transit operation or permit anyone else to use them in this way, and, further, the company retains the right to run over such lines if necessary to reach any shop or yard it may have along such extensions. The franchises for third track on any of the elevated-railway trunks apparently become coterminous with the franchises for the remainder of the structure on which they lie, that is to say, perpetual.

Tests of Locomotive Boilers Having Separated Firebox and Barrel Sections

Comparative tests of locomotive boilers with fireboxes of the standard stay-bolt and the Jacobs-Shupert sectional type (described in ENGINEERING NEWS, June 17, 1909), were noted in detail as projected in ENGINEERING NEWS, Feb. 15, 1912, pages 315-16. These trials have been started and a preliminary progress report of the first series, made under the direction of Dr. W. F. M. Goss, of Urbana, Ill., has been furnished by the Jacobs-Shupert U. S. Firebox Co., from which report the following information has been taken.

Both boilers are identical in general dimensions, and the purpose of the first series of tests was to determine for each boiler the evaporation from the firebox and from the tubes separately. To make such a determination possible, the firebox tube-sheet was extended in all directions to the outside shell, thus forming a diaphragm completely separating the water space on the two sides of this tube-sheet. Each boiler is in effect two—the heating surface of the one comprising all portions of the firebox except the tube-sheet and the heating surface of the other comprising the tubes and tube-sheet.

In carrying out the tests, each compartment was supplied with water as though it were a separate boiler. The quality of steam delivered from each was determined independently and every attempt made to determine with highest possible accuracy the heat delivered through the walls of the firebox and through the flues. The surface dimensions of the two boilers under tests are interesting in this connection:

	Heating surface, square feet	
	Radial-Stay Boiler	Jacobs-Shupert Boiler
In the firebox	179.2	201.9
In the barrel	2805.1	2806.5
Total for both	2984.3	3008.4

A series of tests with oil fuel was run on each boiler, three different rates of power being employed in each series with the fuel consumption ranging from 800 to 2100 lb. per hour. The total water evaporated from both ends has ranged from 10,000 to 24,000 lb. per hour, the evaporation per pound of oil being some 16 lb. at the lower capacity and 14 lb. for the higher.

In all tests a surprisingly large percentage of the total work was done by the firebox, the percentage decreasing for the higher reading. At low rates, from 45 to 50% of the total heat transmitted is absorbed at the firebox. The lowest value secured was approximately 34%. As the heating surface of the firebox is a small fraction of the total heating surface of the boiler, it is evident that heat is transmitted at rates which are extremely high. Results of a number of tests show an evaporation of more than 50 lb. of water per sq.ft. of firebox heating surface per hour, which rate of evaporation is equivalent to the development of 300 hp. by the firebox alone. The firebox was isolated and could not benefit by the circulation of water from the forward end of the boiler backward into the water legs.

The experimental results have not been published in detail, but it is stated that the absorption of heat by the Jacobs-Shupert firebox is somewhat in excess of that absorbed by the radial-stay type, and taking the boilers as a whole the former is slightly more efficient.

A series of coal-fired tests duplicated those with oil fuel. The results obtained, as to the distribution of work between firebox and tubes and the relative performance of the two types of boiler, were in agreement with those obtained with oil.

Dry Calcium Chloride as a Dust Preventive was used by the Boston Street Department in January, 1911, according to its most recent annual report. The chemical was applied when the temperature was below freezing and the streets were "somewhat dry and dusty." About 51,000 sq.yd. were treated at an average cost of 0.36c. per sq.yd. for one application. Two parts of calcium chloride were mixed with one part of sand and the mixture applied with a spreading machine. The result was "satisfactory, but the cost was somewhat high."

140,000-Volt Transmission Line Equipment

The 140,000-volt project of the Eastern Michigan Power Co. was described in our issue of May 16. We give below some additional particulars of the high-tension equipment from information and drawings furnished by the General Electric Co., Schenectady, N. Y.

LIGHTNING ARRESTERS—These are of the aluminum-cell electrolytic type, described in our issue of June 10, 1909, and consist of stacks of aluminum cones mounted in a tank, but differing considerably from arresters which have been used on systems of lower voltage. Their construction is shown in Fig. 1. Because of the great number of cones necessary, it was essential to arrange them in two stacks for each phase, thereby materially reducing the height of the arrester. The cones for each phase are mounted on an insulating support which rests on the bottom of the tank, and the cones and their insulating support can be lifted from the tank by means of flexible cables attached to the base of the support. As the necessary insulation for the cones is placed inside of the tank the latter can be grounded, which is a material advantage from a standpoint of safety.

The electrolyte partially fills the space between the cones, as in all types of aluminum arresters. The remaining space between the cones, as well as the entire tank, is filled with oil which acts as an insulating and cooling medium. Insulating barriers surround the stacks of cones and not only give added insulation but aid in the circulation of the oil. The entrance bushings are of the oil-filled type, somewhat similar to the wall entrance, transformer and oil-switch bushings. Their chief difference is in the construction of the upper section, which is made of porcelain petticoats suitably arranged to shed the water, this being necessary since the arresters are installed out of doors.

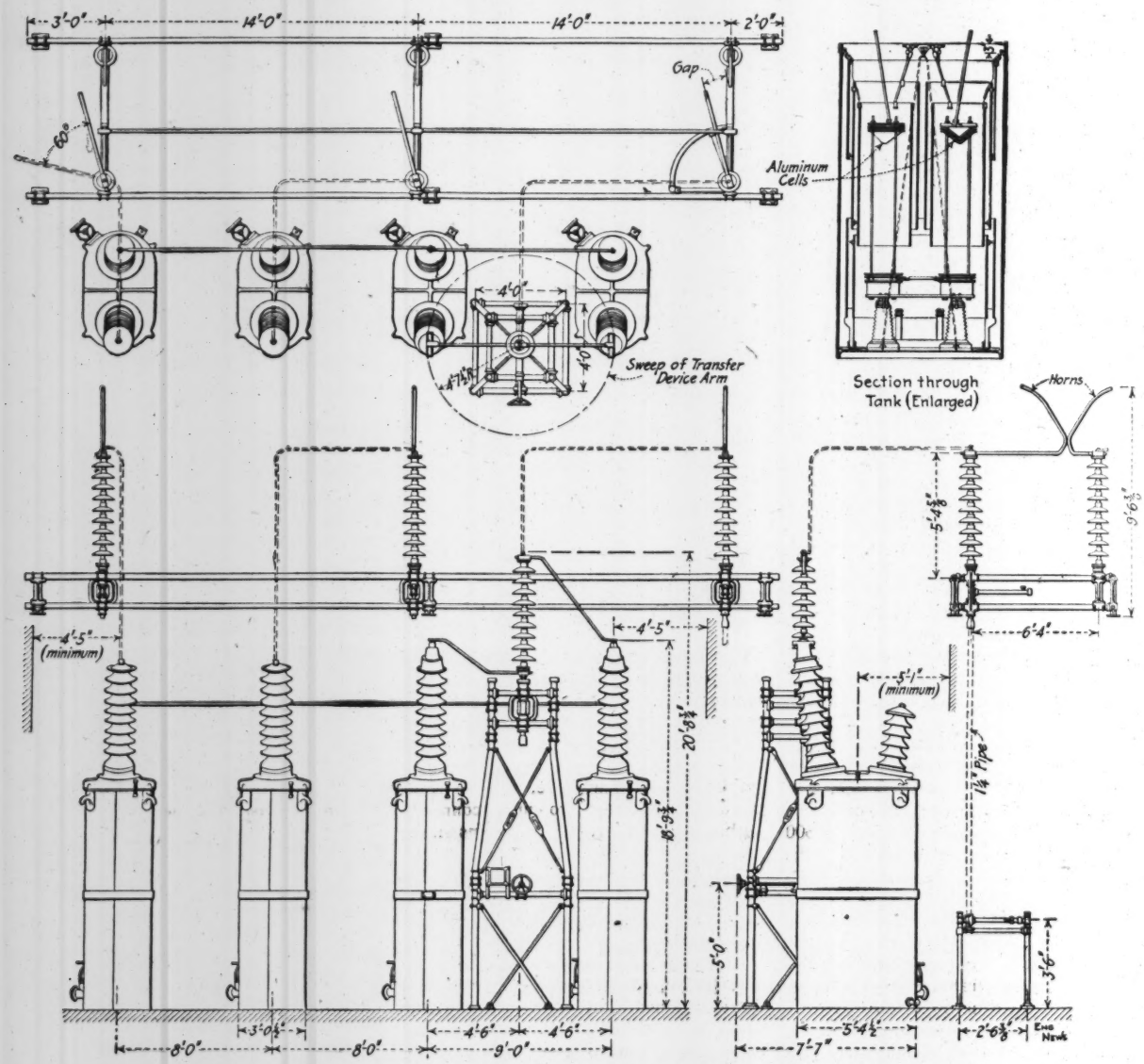


FIG. 1. ALUMINUM-CELL LIGHTING-ARRESTER FOR 140,000-VOLT TRANSMISSION LINE

INSULATING BUSHINGS—Each entrance bushing for the transformer and switch tanks, etc., consists essentially of a conductor surrounded by a semi-fluid insulating compound contained in an insulating shell. The shell is built up of (1) porcelain end pieces, (2) a series of (alternate) non-carbonizing rings and (3) treated barriers; the compound chamber

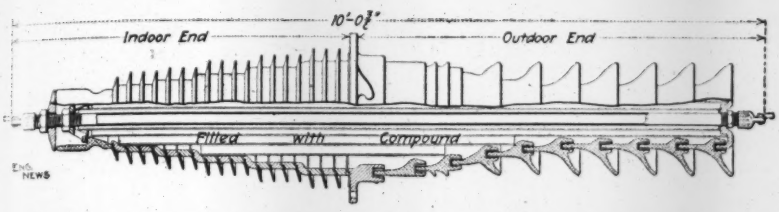


FIG. 2. ENTRANCE WALL BUSHING FOR 140,000-VOLT CONDUCTOR

contains a number of insulating cylinders concentric with the conductor. This is shown by the left-hand half of the bushing seen in Fig. 2. The barriers between rings are of large diameter to prevent surface discharge, thereby increasing the arcing-over voltage. The concentric cylinders in the compound distribute potential strains on the interior of the bushing and increase the rupturing strength. The conductor which passes through the bushing serves also as a clamping bolt and holds together the integral parts of the bushing. About midway between top and bottom of the bushing there is a flanged-iron section which supports the bushing when set in the top of the switch tank and bolted to it. This construction enables the bushing to be easily removed for inspection.

The 140,000-volt wall-entrance bushings (Fig. 2) are similar to the oil-switch bushings except that the portion of the shell outside the station is made of porcelain. They are tested at three times normal operating voltage.

TRANSFORMERS — The design of the main transformers has already been noted (ENG. NEWS, Feb. 15, 1912, p. 315). It will be recalled that they are single-phase 3000-kv.a. types connected in delta to form 9000-kv.a. banks.

At the Owosso substation the transformers are of much more complicated design than usual, on account of embracing some six separate and distinct secondary windings; one for 1000 kv.a. at 5500 volts, one for 1500 kv.a. at 44,000 volts, and four for 125 kv.a. each at 370 volts, so that power may be taken off simultaneously at all of these voltages. Each unit occupies a floor space of 5x12½ ft. and the height over the high-tension leads is 19 ft. These transformers successfully withstood a one-minute high-potential test of 280,000 volts applied between the primary and secondary winding which was grounded to the core, and also successfully withstood an induced voltage test of twice normal stress across the full winding.

OIL SWITCHES — It was particularly necessary that the oil switches should be carefully designed and tested to insure against breakdown, since they had to endure all stresses similar to those borne by the line insulators, transformers, etc., and in addition, those which are added by the necessity of breaking the circuit without undue disturbance under the various abnormal conditions which may arise—that is, they must have an added factor of safety. A high-potential test of 465,000 volts was passed.

These switches are, in general, similar to the hand-operated tank type in use in large power systems here and abroad. Each switch consists of three single-pole separate-tank elements, one in each phase, and in each element there is a double break of sufficient length for more than 140,000 volts. Since it would be impossible to close switches of this size by hand with the ordinary operating mechanism, a newly developed tandem mechanism, in which the operating handle moves through an angle of 180°, is employed. Some of these switches are automatic, while others are not. The former are opened on overload by a series trip, which acts directly on the locking latch.

The Operation of Irrigation Systems

Various phases of the operation of irrigation systems were discussed in November, 1911, at a Conference of Operating Engineers for Irrigation Canal Systems Located in Idaho, Oregon and Washington. The conference was called by request of F. H. Newell, Director of the U. S. Reclamation Service. Fifty invitations were sent to men connected with the larger irrigation systems in the states named, including systems constructed under the Reclamation and Carey acts and by private capital. The conference was held at Boise, with 52 in attendance. F. E. Weymouth, supervising engineer U. S. Reclamation Service, Boise, Idaho, has kindly sent us a mimeographed copy of the Proceedings of the Conference from which we have taken the following notes.

OWNERSHIP AND CONTROL OF THE SMALLER LATERALS — A discussion of some phases of operating irrigation systems, led by Mr. Weymouth, was devoted largely to the ownership of laterals between the main or principal branch canals and the land under irrigation. The "opinion seemed to be unanimous that the only way that a high duty of water could be obtained was to have the central organization operate the laterals to the farm unit."

OPERATING ORGANIZATION—For areas of 20,000 to 100,000 acres, R. K. Tiffany suggested the following schemes of organization:

The general plan of organization which is now in use on the Sunnyside project and which is the result of almost 20 years of experiment with gradual improvements from time to time, is believed to contain the essential principles of such an organization and to be sufficiently elastic to fit the requirements of any size of project. Under this plan the organization for different sized projects would be, approximately, as follows, the number of men provided for being considered the maximum required in any case, while for larger farm units the number of patrolmen and the clerical force could be correspondingly reduced. This organization is based upon the plan of controlling the delivery of each individual farm unit and having reports of individual deliveries of water at intervals of not to exceed a week or ten days, and does not contemplate the visiting of each individual delivery by the patrolman every day:

FOR 20,000 ACRES

	Monthly salary
Irrigation manager	$ 200
Chief clerk and fiscal agent	125
Stenographer and water record clerk	75
Engineer and hydrographer	125
Watermaster	125
8 Patrolmen @ $90	720
Total	$1370

Considering the length of the irrigation season of seven months, this would bring the monthly cost for operation up to 6.8c. per acre, or for the season of seven months about 47c. per acre.

FOR 50,000 ACRES

	Monthly salary
Irrigation manager	$ 250
Chief clerk and fiscal agent	150
Bookkeeper	125
Stenographer	90
Engineer and hydrographer	150
Assistant	100
Water record clerk	90
2 Watermasters @ $150 and $125	275
20 Patrolmen @ $90	1800
Total	$3030

An average cost of about 6c per acre per month, or for an irrigation season of seven months, 42.4c. per acre.

FOR 100,000 ACRES

	Monthly salary
Irrigation manager	$ 300
Chief clerk and fiscal agent	175
Bookkeeper	125
2 stenographers @ $100 and $80	180
Engineer	150
Hydrographer	100
2 Assistants, inspectors, etc., @ $90	180
3 Watermasters, @ $150 and $125	400
40 Patrolmen @ $90	3600
1 Telephone lineman	90
2 Water record clerks @ $90	180
Total	$5480

Or 5½c. per acre per month, or for an irrigation season of seven months, 38.4c. per acre.

It is probable on a great many projects in which the size of the farm unit is 40 to 80 acres, that the number of patrolmen and the clerical force might be reduced from the above figures by about one-fourth and the expense would, of course, be much less for districts where the irrigation season is shorter.

It is believed that the general plan of having three departments is essential to the best operation of an irrigation system.

Accounting Department — The chief clerk and fiscal agent would be at the

head of this department, and would be responsible for all accounts, collections and disbursements, and also for furnishing clear or delinquent lists of water users, in order that the water delivery may be kept, at all times, strictly in accordance with the payments of the water users.

Engineering Department—The head of this department, who on the smaller projects would probably be also the irrigation manager, would be responsible for all work of a technical or engineering nature, including hydrography, designing of structures, keeping of water records, repairs, and improvements of existing canal or structures, and he could also personally or through his assistant, make occasional inspection of the patrolmen's work, instructing them in the proper methods of water measurement and the conditions necessary to insure correct measures through the different measuring devices.

Watermaster's Department—The watermasters would be responsible for the actual handling and distribution of the water to the water users and for the actual work of maintenance of the irrigation system. They should be men of large experience in irrigation work and practice, capable of acting as foremen of earth, timber or concrete work. They should understand the actual application of the water to the land and be familiar with the most improved methods for securing the most economical use of water. They must, of course, be men of tact and courtesy in their dealings with the water users, even tempered and of a judicious turn of mind, for upon them usually devolves the duty of acting as arbitrator between the patrolmen and the water users in case of disputes which may arise regarding water deliveries.

The duties of the inspector provided for in the organization for 100,000 acres, and whose duties will be performed by the engineer or hydrographer on smaller projects, would be to go with the patrolmen as frequently as possible over a portion of his district, observe his methods of water delivery and of keeping records, and give him necessary instructions along these lines.

This topic, as well as the following one, on "Records," brought many detailed suggestions, including organization charts and record blanks.

CONTINUOUS FLOW OR ROTATION METHODS OF WATER DELIVERY—E. B. Darlington, chief engineer and watermaster, Salmon River Project of Twins Falls North Side Land & Water Co., Milner, Idaho, opened the discussion of this subject by stating that his experience had been confined chiefly to one of the two methods in question. His company used the rotation method last summer because a high efficiency was essential, the water being from storage, and because the company's contract with the State Land Board calls for rotation. The continuous-flow system leads to the use of water whether needed or not. For small holdings, such as 5 or 10 acres, the contract allowance of water would give insufficient head for efficient irrigation. F. W. Hanna, project engineer, U. S. Reclamation Service, Boise, Idaho, and others agreed with Mr. Darlington in favoring the rotation method, but some speakers took the opposite side. For large areas and considerable heads wastage of water and damage through erosion, it was urged, would result from the rotation system.

ADJUSTMENT OF COMPLAINTS OF WATER USERS—J. M. Griffin, superintendent of irrigation, Umatilla Project, U. S. Reclamation Service, Hermiston, Ore., based his remarks on experience gained on that project, where most of the settlers are new to farming by irrigation and some of them are also "new to any problems connected with farm life and practice in a raw desert country." There are two classes of settler. One is the speculator, with comparatively large holdings but with little land under irrigation, many of whose complaints are biased by his speculative interests. The other class of complainer is the real settler, whose hopes and interests are bound up with the reclamation of his land. Needless to say, it is the second class which deserves chief consideration. Their complaints, properly studied, may serve as guiding lights.

In dealing with complaints of lack of water Mr. Griffin said, in conclusion:

> To sum up, the speaker believes that in the matter of the adjustment of the lack of water complaint the effort should be to give the settler the water he needs for beneficial use and no more; to stop waste and constantly inculcate the principles of economical use of water. In order to do this, it will require painstaking observation and study on the part of the management to learn what the measure of beneficial use is. First know what is needed and then the measures for securing the end will be easily enough found.

In contrast, there are also complaints of too much water. These may be obviated by preventing over-use of water on higher lands and by drainage.

Breaking of ditches subject to risk through unfavorable local conditions may be largely prevented and money saved in the long run by more permanent construction, as is indicated by the "10 to 14 miles of concrete pipe lines and cement-lined ditches [which have been used] through two seasons on otherwise unsafe lines."

ECONOMIES IN THE USE OF WATER—A paper on this subject by F. W. Hanna, of Boise, Idaho, reviewed the question of seepage and evaporation in relation to the duty of water. A seepage-loss table by Saville, given in the 1908 report of the Isthmian Canal Commission, was quoted, as also data from Sunnyside Canal (Washington) experiments, given by J. C. Stevens in the *Transactions* of the American Society of Civil Engineers. Tables of percolation data and of duty of water tests on various crops obtained recently by the U. S. Reclamation Service on the Boise project, Idaho, were also given. Mr. Hanna summed up certain of his conclusions as follows:

> From the data on losses herein presented, it appears that roughly 40% of the water diverted for irrigation is lost in transit and 30% more in the farm fields. This makes a total loss of 70%, and a total utilization of only 30% of the quantity diverted, which may be considered the approximate water efficiency of the average large irrigation system. With an assumed average gross crop yield of 4¼ tons per acre and a crop yield of one ton to 450 tons of plant use of water, the crop requirement of water per acre is about 1.4 acre-feet. On the basis of the percentages of losses above given this crop yield and plant use of water corresponds to a duty of water of 2.8 acre-feet at the farm headgate and of 4.7 acre-feet at the point of diversion. With larger crop yields a larger quantity of water per acre would evidently be required for successful agriculture.
>
> Of the 70% of water estimated as lost in irrigation systems, about 50% escapes over and into the ground, causing the ground water to rise, resulting in injury to the lower lands, or in the construction of expensive drainage works. Hence this extravagant use of water is a detriment to the user as well as to the advance of irrigation development.

Some of the other papers presented at the conference have been reserved for possible future separate treatment.

A Long-Range Blasting Accident occurred, May 20, at Tuxedo Park, N. Y., when a man standing near his home was struck and killed by a 20-lb. rock hurled about 1200 ft. by a blast on the new state road.

An Escalator Accident in New York City at the 177th St. station of the Interborough subway, May 26, caused the injury of 20 persons. From the accounts of the accident in the daily press, it appears that the chain supporting the outer edges of the escalator steps broke, allowing the steps to flatten out. The passengers slipped down and were heaped together at the foot of the incline.

A Railway Bridge Collapsed, May 18, dropping nine loaded freight cars into the Firesteel River near Mass City, Mich. Reports state that the locomotive and first two cars crossed the bridge in safety. The following nine cars, loaded with rock or ore, fell through the bridge into the river 60 ft. below, while the two passenger coaches and caboose at the rear end of the train remained on the track. No one was injured. The bridge was on the line of the Mineral Range R.R.

A Gang Plank Dropped, May 19, at the Coleman dock at Seattle, Wash., precipitating 60 passengers, who were about to board a steamboat, into the water. About 40 were more or less hurt and two persons were drowned. The vessel was the "Flyer," a sound steamer plying between Seattle and Tacoma. Press reports state that the accident was due to a defective cog in the gear by which the gang-plank was raised and lowered. The shock of the 1-in. drop of the loaded plank due to the failure of this cog was sufficient to strip the other teeth from the gear and let the end of the plank fall.

An Automobile Fell Through a Drawbridge into the Calumet River at 92d St., Chicago, Ill., May 20. Five people in the car were drowned, only one of them even coming to the surface after the accident. Report states that the automobile approached the bridge at a speed

of 30 mi. per hr. The bridge had just been swung open to allow a boat to pass, and a warning shouted to the chauffeur was not in time to prevent the accident. Witnesses testified that the car turned completely over as it went into the river.

The S.S. "Imperator" of the Hamburg-American line was successfully launched, May 23, from the Vulcan yard, at Hamburg, Germany, and was christened by the Kaiser. The new ship has a registered tonnage of 50,000, as compared with the 45,000 gross tons of the "Olympic," and will have a displacement when completed of about 70,000 tons. Her length over all is 900 ft., beam, 96½ ft. She will have four screw propellers driven by turbine engines of 70,000 hp., designed to give her a speed of 22½ knots per hour.

The "Imperator" will have accommodations for 4100 passengers. Lifeboat and liferaft capacity will be provided sufficient to carry every one of this number together with her crew of 1100. The type of davits and the special hoisting and lowering apparatus to be used for her lifeboats was described in Engineering News, May 23, 1912, p. 978.

This vessel will be the first of anywhere near her size to be equipped with the Frahm "U" tanks for the prevention of rolling. The Frahm tanks (Eng. News, Feb. 9, 1911, p. 181) were tried out in 1910 on the Hamburg-American liners "Ypiranga" and "Corcovado," notoriously heavy rollers, and the installation of the Frahm apparatus on the "Imperator" was decided upon as a result of these trials.

When Is a Railroad not a common carrier, recently has been decided by the Interstate Commerce Commission in the so called "Tap-Line Case." The commission for several years has been investigating those short stretches of railway line which were constructed primarily to serve some particular industry, in the general interest of which it is owned and operated. All of the lines which figure in the present case are lumber roads into the forest and from mills to trunk-line railways. At one time the rails, tracks and locomotives were very largely owned and operated by the lumber concern itself and no pretence was made of serving outside interests. In recent years, however, it has become a common practice to turn the "tap-line," from mill to trunk railroad, over to a separate company, and these have been dealt with by the trunk line as wholly apart from the industry and as if they constituted a separate common carrier in general public service. Many of these lumber lines receive allowances out of the rate charged by the trunk line for interstate shipments and figured from the forest rather than the mill, although the lumber concerns still held the logging spurs to monopolize the timber near the mills. The Commission estimates that such allowances amount to not less than $60,000,000 per year. The numerous complaints of discrimination arising out of such relations were the cause of the Commission's investigation.

The Commission has now ruled that although the rails, locomotives and cars may have been turned over to an incorporated railroad company owned and operated by the industry, or else in its interest, does not divest the equipment of status as a plant facility. It holds on the other hand that the common ownership of an industry and a short railway line serving it is not sufficient to divest the railroad of status as a common carrier, but that the separate cases must stand on their own facts. The Commission intimates that it will permit divisions of rates in cases where the tap line is an actual common carrier, but that the amount received shall bear some definite relation to the service rendered, to eliminate the species of discrimination found to exist.

The U. S. Battleship "Texas," the largest warship yet constructed for the U. S. Navy and also the largest in armament in the world, was launched at Newport News, Va., on May 18. The "Texas" has an overall length of 573 ft., a breadth at the water line of 95 ft. 2½ in., and a displacement under full load of 28,300 tons. She is driven by twin screws with vertical triple expansion reciprocating engines, having about 27,000 hp. The estimated speed is 21 knots. Her armament includes ten 14-in. guns mounted in five turrets, twenty-one 5-in. rapid-fire guns besides the smaller equipment.

On the same day there was launched in England the new Japanese battle cruiser "Kongo." This ship displaces 27,500 tons, carries eight 14-in. and sixteen 6-in. guns, and is fitted with turbine engines designed to give her a speed of 28 knots.

The Commission Plan of City Government has been adopted by St. Paul, Minn., and by Atlantic City, N. J. St. Paul, with its population of about 215,000 at the last census, is the largest city which has yet adopted this plan. The six other cities of more than 100,000 population which have adopted the commission plan, together with their population and date of adoption, are as follows: Lowell, Mass., 106,294; Birmingham, Ala., 132,685; Memphis, Tenn., 131,105; Omaha, Neb., 124,096; Spokane, Wash., 104,402; Oakland, Calif., 150,174. Memphis adopted the plan in 1909; Spokane and Oakland, in 1910; the other three cities in 1911. On the same day that Atlantic City adopted the plan (by a vote of 3304 for and 3182 against), Elizabeth, N. J., voted against it.

Accidents on the Rapid Transit Lines in New York City appear to be on the increase. During April, 1912, according to the report of the Public Service Commission, the number of persons injured on the elevated, subway and surface railroads of the city was 3310, an increase of 474 over the number of persons injured in April, 1911. Of those injured 22 were fatally hurt. Nearly half of the total number of persons injured was made up of passengers who sustained injuries while getting on or off cars. The exact figures are 725 hurt while boarding cars and 610 thrown down and injured in alighting from cars.

Personals

Mr. J. W. Callahan, formerly Assistant Superintendent of the Indiana Harbor Belt R.R., at Gibson, Ind., has been made General Manager of the Illinois Tunnel Co., Chicago, Ill.

Mr. P. O. Walker, formerly Roadmaster of the Missouri, Kansas & Texas Ry., at Paola, Kan., has been appointed Roadmaster of the Atchison, Topeka & Santa Fé Ry., at Guthrie, Okla.

Mr. C. A. Allen, formerly Superintendent of the Erie R. R. at Marion, Ohio, has been promoted to be Assistant General Superintendent with office at Galion, Ohio. Mr. R. E. Woodruff, Superintendent at Rochester, N. Y., succeeds Mr. Allen at Marion.

Dr. Arthur E. Haynes, Professor of engineering mathematics at the University of Minnesota, has been retired at his own request because of physical disability. Prof. Haynes is 63 years old and has taught for 37 years, 16 of which have been at the University of Minnesota.

Mr. George B. Hull, M. Am. Soc. C. E., Assistant Engineer, Department of Public Works of Canada, who had charge of the surveys for and construction of the Quinze dam of the Upper Ottawa River reservoir system, has been appointed District Engineer for the Department of Public Works in British Columbia, with headquarters at Prince Rupert.

Mr. Joseph H. Young, formerly President of the Alaska Steamship Co., and Vice-President of the Copper River & Northwestern Ry., has been elected President of the Spokane, Portland & Seattle and the Oregon Trunk Rys., with offices in Portland, Ore. Mr. Young was born in Salt Lake City, Utah, in 1864, and in 1881 began railway work as a clerk with what is now the Oregon Short Line R.R. He served in the passenger traffic departments of a number of western railways and in 1904 became General Superintendent and later General Manager of the Colorado & Southern Ry. at Denver, Colo. From 1907 to 1910, Mr. Young was General Superintendent of the Southern Pacific Ry., at San Francisco, Calif.

Mr. E. J. Chamberlain, former Vice-President of the Grand Trunk Pacific Ry., and since the death of C. M. Hays, acting President, has been chosen President of the Grand Trunk Ry. Co., of Canada, succeeding Mr. Hays in the management of all the Grand Trunk lines. Mr. Chamberlain was born in Lancaster, N. H., and entered into railway work in 1871 as a timekeeper in the car shops of the Central Vermont R.R., at St. Albans, Vt. He then served as a clerk until 1877 when he was made private secretary to the General Manager, a position he held for seven years. In 1884 he was made Superintendent of the Ogdensburg & Lake Champlain R.R. and Central Vermont Line steamers running between Ogdensburg and Chicago, Ill. From 1886 to 1905 he was General Manager of the Canada Atlantic Ry., which became a subsidiary of the Grand Trunk Ry. Since 1905 he has been an executive officer of the Grand Trunk and Grand Trunk Pacific Rys.

Obituary

Sir Julius Charles Wernher of Wernher, Beit & Co., of London, England, and prominent in the development of the South African diamond mines, died on May 21.

B. L. Buck, a civil engineer employed by the West Penn. Rys. Co., of Pittsburg, Penn., died on May 20, aged 27 years. For about four years he was with the Pennsylvania State Highway Department.

Thomas Burgess, one of the founders and for many years Superintendent of the Vulcan Iron Works, of Chicago, Ill., died on May 19, at the age of 79 years. He was one of the first to engage in the

manufacture of dredging machinery in the West.

Henry M. Rogers, who was contractor for many wharves and piers on the North River front of New York and Jersey City, died on May 24, at his home in Huntington, Long Island, N. Y. He retired from active business more than 20 years ago. He was born at Huntington in 1826.

J. M. Lee, Chief Engineer of the Corporation Commission of Oklahoma, died on May 20. He was formerly Superintendent of the San Antonio Branch division of Galveston, Harrisburg & San Antonio Ry., at San Antonio, Tex., and at one time was Vice-President and General Manager of the Antonio & Gulf R.R.

Joseph Lewis Dixon, a mining engineer employed by the Ingersoll-Sargent Co., of Pittsburg, Penn., died on May 23, aged 49 years. He was born in England and came to this country when 17 years old. After working in the West Virginia mines for a number of years he accumulated enough money to obtain a technical education in mining engineering at the Ohio State University. For a number of years he was Superintendent of the Pinnicknick Coal Co., at Clarkesburg, W. Va. He was also a newspaper writer of note.

Valdemar F. Lassöe, a prominent engineer and naval architect of New York City and for many years consulting engineer of the American-Hawaiian Steamship Co., died at his home in Brooklyn, N. Y., on May 22. He was born in Copenhagen, Denmark, in 1836, and was educated as a civil engineer. He came to America at the age of 24 years, and was associated with John Ericsson until the latter's death in 1889. Mr. Lassöe also served for a number of years as Consulting Engineer in America of the North German Lloyd Steamship Co., and was a well known member of the Society of Naval Architects and Marine Engineers.

Engineering Societies

COMING MEETINGS

AMERICAN WATER WORKS ASSOCIATION.
June 3-8. Annual convention at Louisville, Ky. Secy., J. M. Diven, 271 River St., Troy, N. Y.

ASSOCIATION OF RAILWAY TELEGRAPH SUPERINTENDENTS.
June 4. Annual convention at New York City. Secy., P. W. Drew, Soo Line, Chicago, Ill.

NATIONAL ELECTRIC LIGHT ASSOCIATION.
June 10-14. Annual meeting at Seattle, Wash. Secy., T. C. Martin, 29 West 39th St., New York.

MASTER CAR BUILDERS' ASSOCIATION.
June 12-14. Annual convention at Atlantic City, N. J. Secy., Jos. W. Taylor, 390 Old Colony Bldg., Chicago.

AMERICAN RAILWAY MASTER MECHANICS' ASSOCIATION.
June 17-19. Annual convention at Atlantic City, N. J. Secy., Jos. W. Taylor, 390 Old Colony Bldg., Chicago.

CANADIAN ELECTRICAL ASSOCIATION.
June 19-21. Annual meeting at Ottawa, Ont. Secy., T. S. Young, 220 King St., West, Toronto, Ont.

AMERICAN INSTITUTE OF ELECTRICAL ENGINEERS.
June 25-28. Annual convention at Boston, Mass. Secy., F. L. Hutchison, 33 West 39th St., New York.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
June 25-28. Annual convention at Seattle, Wash. Secy., C. W. Hunt, 220 West 57th St., New York.

SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION.
June 26-28. Annual meeting at Boston, Mass. Secy., H. H. Norris, Cornell University, Ithaca, N. Y.

SOCIETY OF AUTOMOBILE ENGINEERS.
June 27-29. Summer meeting at Detroit, Mich. Secy., C. F. Clarkson, 1451 Broadway, New York.

NATIONAL MUNICIPAL LEAGUE.
July 8-12. Annual meeting at Los Angeles, Calif. Secy., C. R. Woodruff, North American Bldg., Philadelphia, Penn.

AMERICAN SOCIETY OF HEATING AND VENTILATING ENGINEERS.
July 11-12. Summer meeting at Detroit, Mich. Secy., W. W. Macon, 29 West 39th St., New York.

OHIO ELECTRIC LIGHT ASSOCIATION.
July 16-19. Annual convention at Cedar Point, Ohio. Secy., D. L. Gaskill, Greenville, Ohio.

National Building Trades and Employers Association—This organization consists of city, state and inter-state associations connected with the building trades. Its object is declared to be the promotion and protection of the interests of the building trades in the United States. The secretary is Mr. I. H. Scates, 15 E. Fayette St., Baltimore, Md.

American Road Builders Association—A dinner will be given at the Hotel Astor, New York City, June 5, in honor of W. Rees Jeffreys, secretary of the road board of England and honorary general secretary of the local organizing committee of the Third International Road Congress, which will be held in London in June, 1913. Mr. Jeffreys is coming to America to arouse interest in the London Road Congress, which will be the first international road congress to be held in an English speaking country.

International Congress of Applied Chemistry—A supplementary bulletin has just been issued giving the titles of 148 additional papers received since the publication of the last bulletin (issued Apr. 9, 1912), to be presented at the eighth congress at Washington and New York, Sept. 4-13. The total number of papers now definitely promised for the 24 sections of the congress is 683. Among the papers named in this most recent bulletin are the following: "The Methods of the U. S. Steel Corporation for the Commercial Sampling and Analysis of Pig Iron," J. M. Camp, Munhall, Penn.; "Detailed Design and Construction of Modern Wood Preserving Plants," G. B. Shipley, Pittsburg, Penn.; "The Development of the Wood Preserving Industry in the United States," E. A. Sterling, Philadelphia, Penn.; "Corrosion Tests of a Series of Iron Alloys," James Aston, Madison, Wis.; "Heat Losses in Furnaces," F. A. J. Fitzgerald, Niagara Falls, N. Y.; "Influence of Cinders on Corrosion of Imbedded Iron," Walter B. Schulte, Madison, Wis.; "Power Supply Characteristics of the Electric Furnace," Charles P. Steinmetz, Schenectady, N. Y.

National Association of Manufacturers—The 17th annual meeting was held May 20-22 at the Waldorf Astoria Hotel, New York City. Much attention was given to the subject of accident prevention and workmen's compensation. One of the features of the convention was an exhibition of photographs of models of safety methods and devices. There was also a series of motion pictures, showing the cause and effect of a serious industrial accident, illustrating the use of safety devices in a manufacturing plant, and showing a number of factory fire drills in industrial plants in New Jersey. A fourth film showed a lifeboat drill on an ocean liner. The report of the committee on industrial education called attention to the fact that half of all American children leave school at the end of the sixth grade, or at about 14 years of age. Among the several addresses was one by Franklin H. Wentworth, Secretary of the National Fire Protection Association, who pointed out that the fire waste in this country acts as an indirect taxation on everything we eat, drink and wear.

Mr. John Kirby, Jr., was reelected president of the association and Mr. A. B. See was elected treasurer to fill the vacancy caused by the death of F. H. Stillman.

New York Electrical Society—A lecture on "Wave Motion" was given at the meeting of May 15, at the Engineering Societies' Building, in New York City, by Dr. Charles P. Steinmetz. Dr. Steinmetz stated that progress in discovery is always from the special to the general; that is, individual phenomena are first observed and afterward the general law covering a number of related phenomena is formulated. Thus some phenomena which when discovered appear to be something entirely new are seen later to be only special cases under some general law already well established. The alternating-current transformer, he stated, when first discovered was looked upon as an entirely new invention, but it is now hard to realize that anyone familiar with such things could fail to see that the principle involved is identical with that of the ordinary induction coil. In both cases an electric current, produced in one conductor, gives rise to an energy wave in the ether which in turn "induces" a current in another conductor entirely insulated electrically from the first. Doctor Steinmetz pointed out that the only differences between the ordinary medical coil, the alternating-current transformer and the induction motor are such modifications as are naturally due to the different purposes for which they are intended, the principle in all cases being the same.

He went on to prophesy indirectly that the time would come when wireless telegraphy would be looked upon as simply another modification or application of the same principle. The difference was simply one of wave length or frequency. The higher the frequency the greater was the proportion of the energy of the original current which was converted into an energy wave in the ether and the smaller the proportion transmitted as an electrical current through the conductor. The high frequency of the wireless telegraph was accompanied by comparatively short wave lengths of 1000 ft. more or less, while the relatively low-frequency waves of alternating-current transmission lines had a length of some thousands of miles. The lecturer then proceeded to state that waves of frequencies many times that of wireless telegraph waves could be detected by the nerves of the hands, for example, for a distance of some feet from their source. Such waves, he stated, could be readily produced, and in fact were given out by every steam boiler. Waves of even shorter length and higher frequency than these (which we call heat waves) were recognized as light.